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US Army Corps
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Waterways Experiment
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Updated Wave Response of Proposed Improvements to the Small Boat Harbor at Maalaea, Maui, Hawaii

by Lori L. Hadley, Edward F. Thompson, Donald C. Wilson

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Final report

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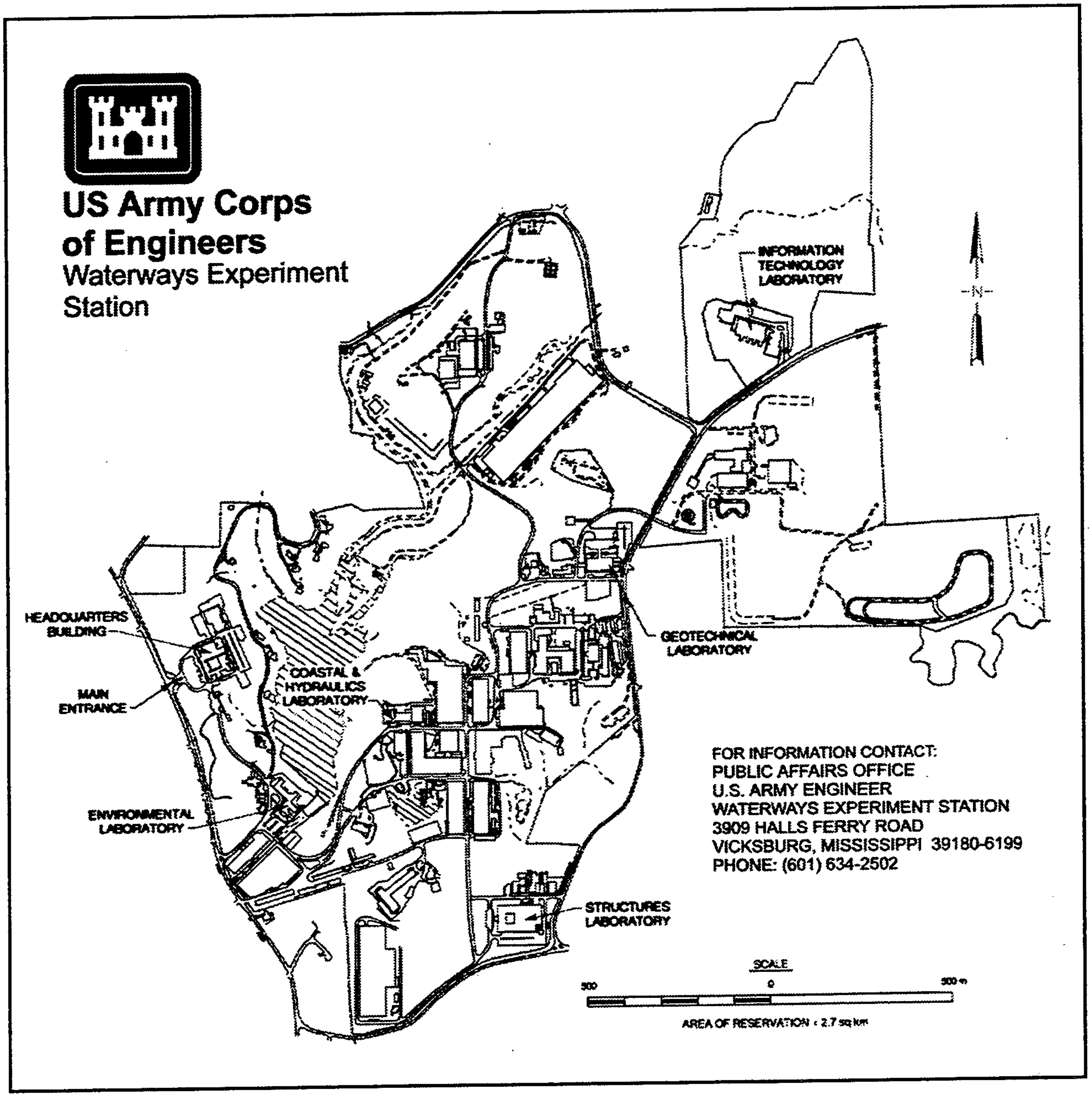

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Preface

This study was authorized by the U.S. Army Engineer Division, Pacific Ocean (POD), and was conducted by personnel of the Navigation and Harbors Division (NHD), Coastal and Hydraulics Laboratory (CHL), of the U.S. Army Engineer Waterways Experiment Station (WES). The study was conducted during the period October through December 1997. Mr. Stanley Boc, POD, oversaw progress of the study.

This report was prepared by Ms. Lori L. Hadley and Dr. Edward F. Thompson, Coastal Hydrodynamics Branch (CHB), NHD; and Mr. Donald C. Wilson, Navigation Branch, NHD. The work was performed under the direct supervision of Dr. Martin C. Miller, Chief, CHB, and Mr. C. E. Chatham, Jr., Chief, NHD, and under the general supervision of Mr. Charles C. Calhoun, Jr., Assistant Director, CHL, and Dr. James R. Houston, Director, CHL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
degrees (angle)	0.01745329	radian
feet	0.3048	meters
knots (international)	0.5144444	meters per second
miles (US Statute)	1.6093	kilometers
nautical miles	1.852	kilometers

1 Introduction

Background

At the request of the U.S. Army Engineer Division, Pacific Ocean (POD), a numerical model wave response study of proposed improvements to Maalaea small boat harbor was conducted by the U.S. Army Engineer Waterways Experiment Station's (USAEWES) Coastal and Hydraulics Laboratory (CHL). The study was conducted as a revision and extension of previous studies (Lillycrop et al. 1993, Thompson and Hadley 1994b) to assess the wave response of various alternative modification plans for the harbor. This report is focussed on the determination of an optimal design plan which would provide the harbor with adequate protection from the incident wave climate. Information provided in earlier reports is referenced in this report, but generally not repeated.

Procedures and methods for conducting this study improve on those used by Lillycrop et al. (1993) in several important ways. The most significant improvement is the deepwater wave estimates used in the study. In previous studies deep water wave estimates were based on measurements in the Monitoring of Completed Coastal Projects program collected at Barbers Point, Oahu. For this study, incident wave data were obtained from National Data Buoy Center (NDBC) Station 51027, a deepwater buoy located southwest of the island of Lanai. The availability of deepwater data nearer the vicinity of Maalaea Harbor significantly improves the validity of the overall results. The current study also incorporates improved model technology. Since initial studies were conducted, spectral wave modeling capabilities for wind waves and swell have been added to the model and, as part of a Coastal Modeling System (CMS) update, several harbor modeling parameters have been investigated and optimized (Lillycrop and Thompson 1996). These adjustments have a notable impact on model performance and have been included in the new Maalaea Harbor study. The current study also provides a complete long wave evaluation for each harbor plan as well as a navigation evaluation based on recent research.

Study Location

Maalaea small boat harbor is located on the southwest coast of the island of Maui, HI, the second largest island in the Hawaiian chain (Figure 1). The

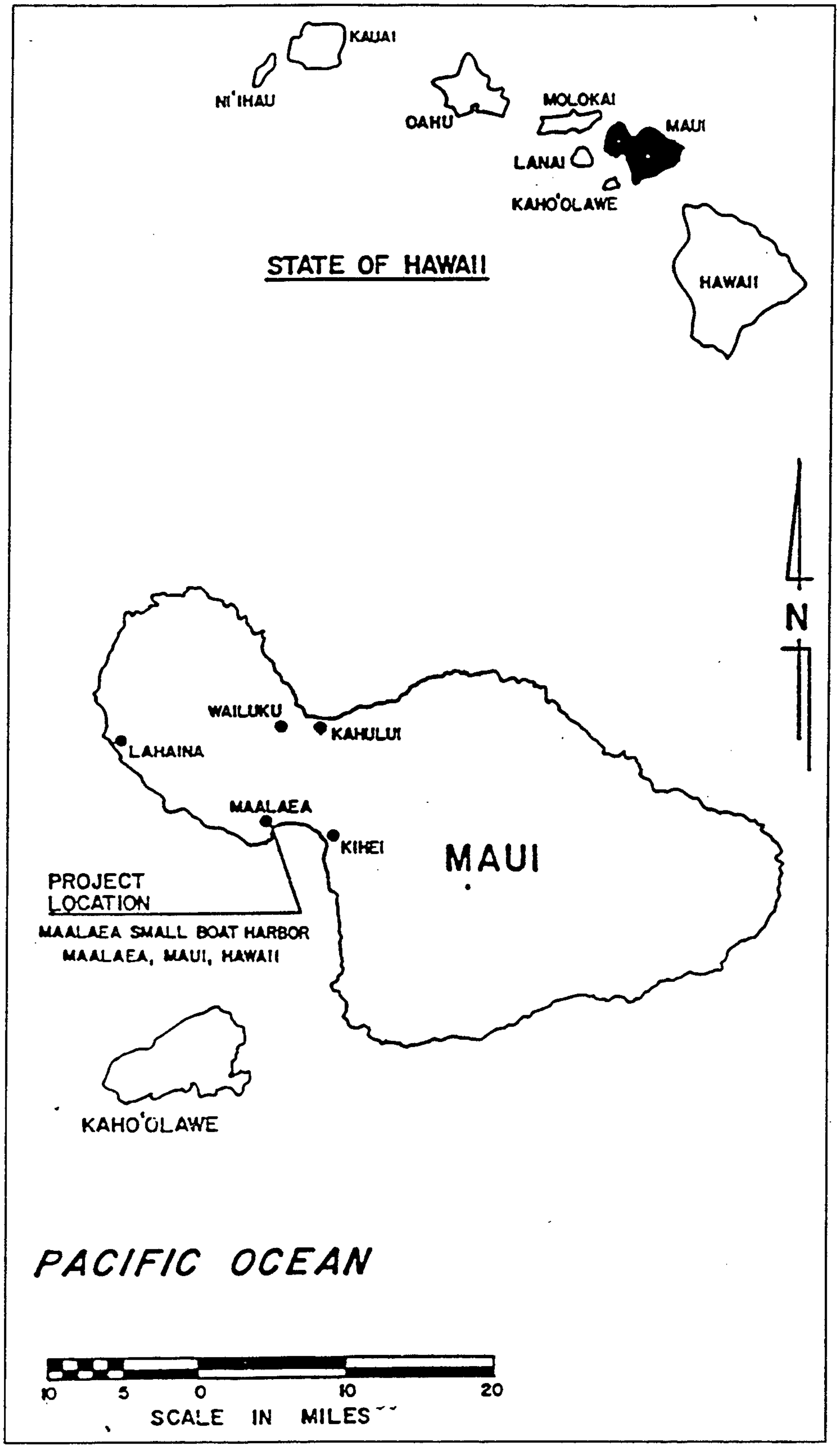

Figure 1. Study location

harbor is approximately 7 miles[1] south of the County seat in Wailuku and approximately 8 miles south of the commercial and business center of Kahului.

Harbor space on Maui is much in demand. Maalaea small boat harbor contains 93 berths. Wave energy penetrates inside the harbor sufficiently often and with enough energy that the harbor is regarded as having a "surge" problem. A larger, more protected small boat harbor at Maalaea would help satisfy the demand for tranquil berthing space. The existing harbor layout is shown in Figure 2.

The shoreline of Maalaea Bay is part of an isthmus connecting two inactive volcanos which form west and east Maui. The shoreline is characterized by a long narrow coral-sand beach. The area is also known among surfers as the Maalaea Pipeline because of an infrequent, but world class breaking wave condition. Maalaea Harbor is located at the extreme west end of this beach. Several lesser surfing spots are also located near the harbor. There is concern that changes at Maalaea small boat harbor may impact nearby surfing areas.

Proposed improvements to Maalaea harbor are limited by several factors. The most significant is that the harbor site is fixed and can not be moved to a more ideal location. Additional considerations arise from recommendations provided by harbor users and local surfers. These recommendations include keeping the existing breakwater structures intact with any changes being additive, constructing modifications without serious interruption to harbor navigation, and limiting additional structures to the present eastern boundary of the harbor in order to avoid impacts on the surfing area outside the harbor. The General Design Memorandum (GDM) for Maalaea Harbor for Light-Draft Vessels (US Army Engineer District, Honolulu 1980) contains a record of the research and planning which led to proposed design improvements, Plan 1 (Figure 3). Plan 1 was subsequently followed by the development of additional modification plans. Plans selected for evaluation in this study are described below.

Plan 1 will provide berthing facilities for approximately 310 small craft, and includes the following improvements:

a. A 620-ft-long extension to the existing south breakwater.

b. An additional 400-ft-long revetment on the seaward side of the existing south breakwater.

c. A 610-ft-long entrance channel, varying in width from 150 to 180 ft, and varying in depth from 12 to 15 ft.

d. A 1.7-acre, 12-ft-deep turning basin.

[1] A table of factors for converting non-SI units of measurement to SI units is provided on page vii.

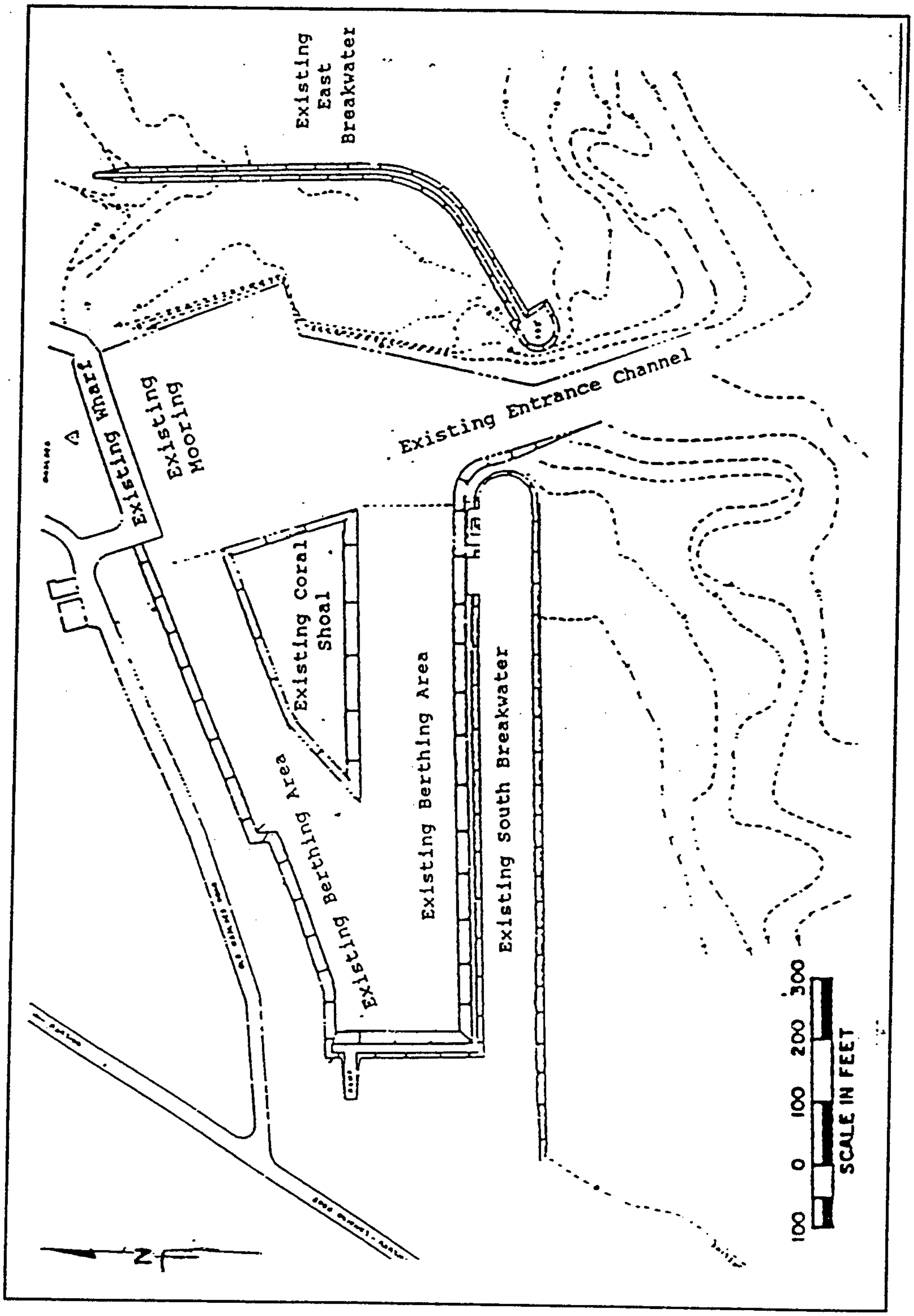

Figure 2. Existing plan

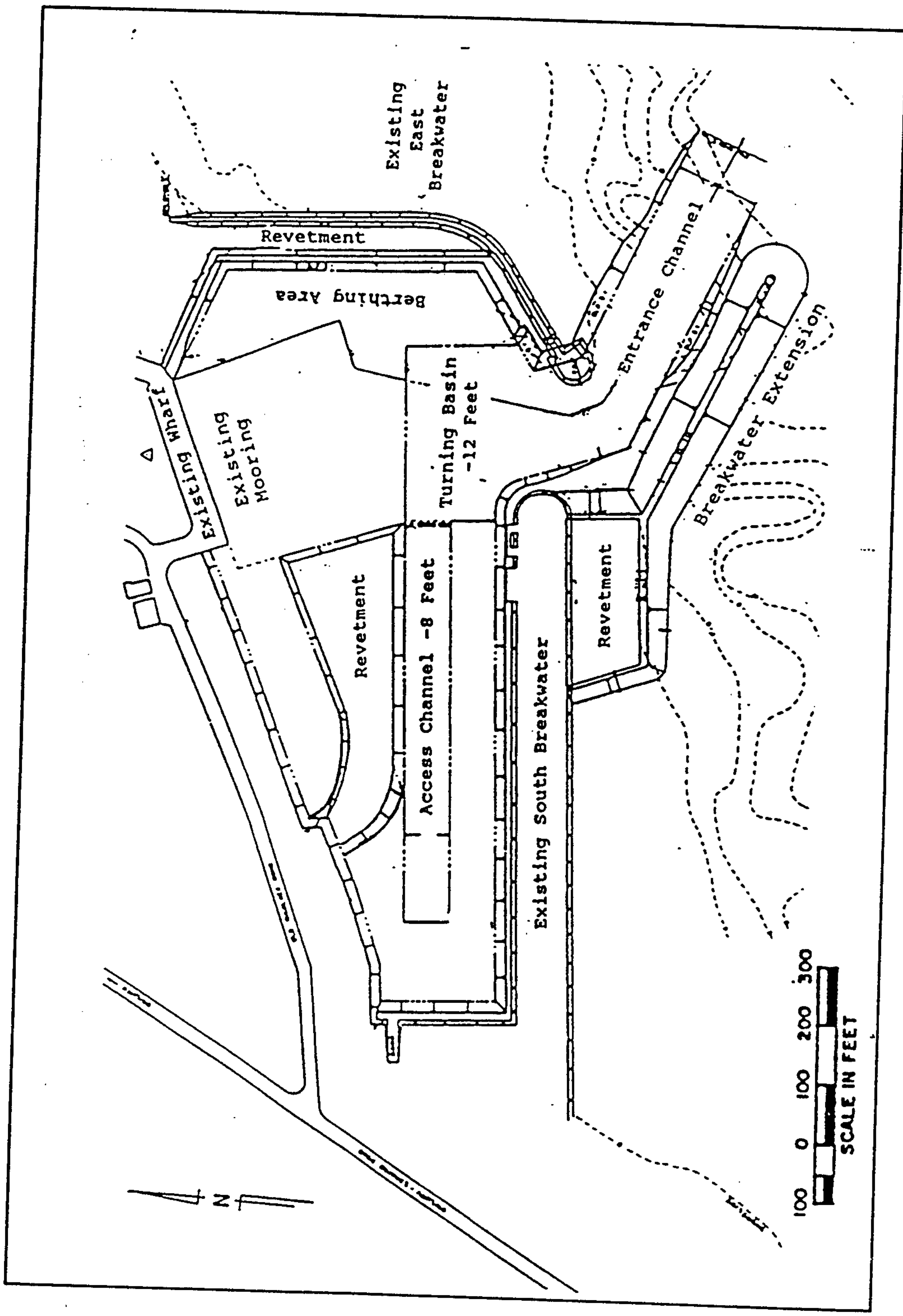

Figure 3. Proposed Plan 1

e. Removal of 80 ft from the existing east breakwater head.

f. A 50-ft-wide, 720-ft-long interior revetment adjacent to the existing east breakwater.

g. An 8-ft-deep berthing area adjacent to the existing east breakwater.

h. A 570-ft-long interior revetment varying in width from 50 to 170 ft.

Plan 2 (Figure 4) redirects the entrance channel to the west and includes the following improvements:

a. Removal of 300 ft from the existing south breakwater tip.

b. A 610-ft-long 15-ft-deep entrance channel, varying in width from 150 to 200 ft, and varying in depth from 12 to 15 ft.

c. A 1.7-acre, 12-ft-deep turning basin.

d. Removal of 80 ft from the existing east breakwater head.

e. A 600-ft-long extension to the existing east breakwater.

f. A 50-ft-wide, 600-ft-long interior revetment adjacent to the existing east breakwater.

g. An 8-ft-deep berthing area adjacent to the existing east breakwater.

h. A 570-ft-long interior revetment varying in width from 50 to 170 ft.

Plan 3 (Figure 5) includes the same improvements as Plan 2 with the exception of an additional extension to the existing east breakwater. The 600-ft-long extension will continue an additional 250 ft toward the west.

Two modifications of Plan 1 were also considered (Figure 6). Plan 1a is the same as Plan 1 except the new south breakwater extension and entrance channel are rotated clockwise 7 deg. Plan 1b is identical to Plan 1a except a vertical sheet pile bulkhead replaces the revetment along the east side of the center mole.

Plan 6 (Figure 7), was added as an alternative for a more protected harbor area without new structures *exterior* to the existing harbor. Its disadvantages include lack of needed new mooring space and a possibly difficult entrance channel section confined between two rock-faced structures. Plan 6 includes the following improvements:

a. Addition of a 95-ft-wide, 500-ft-long mole extending from the east end of the existing south breakwater into the harbor.

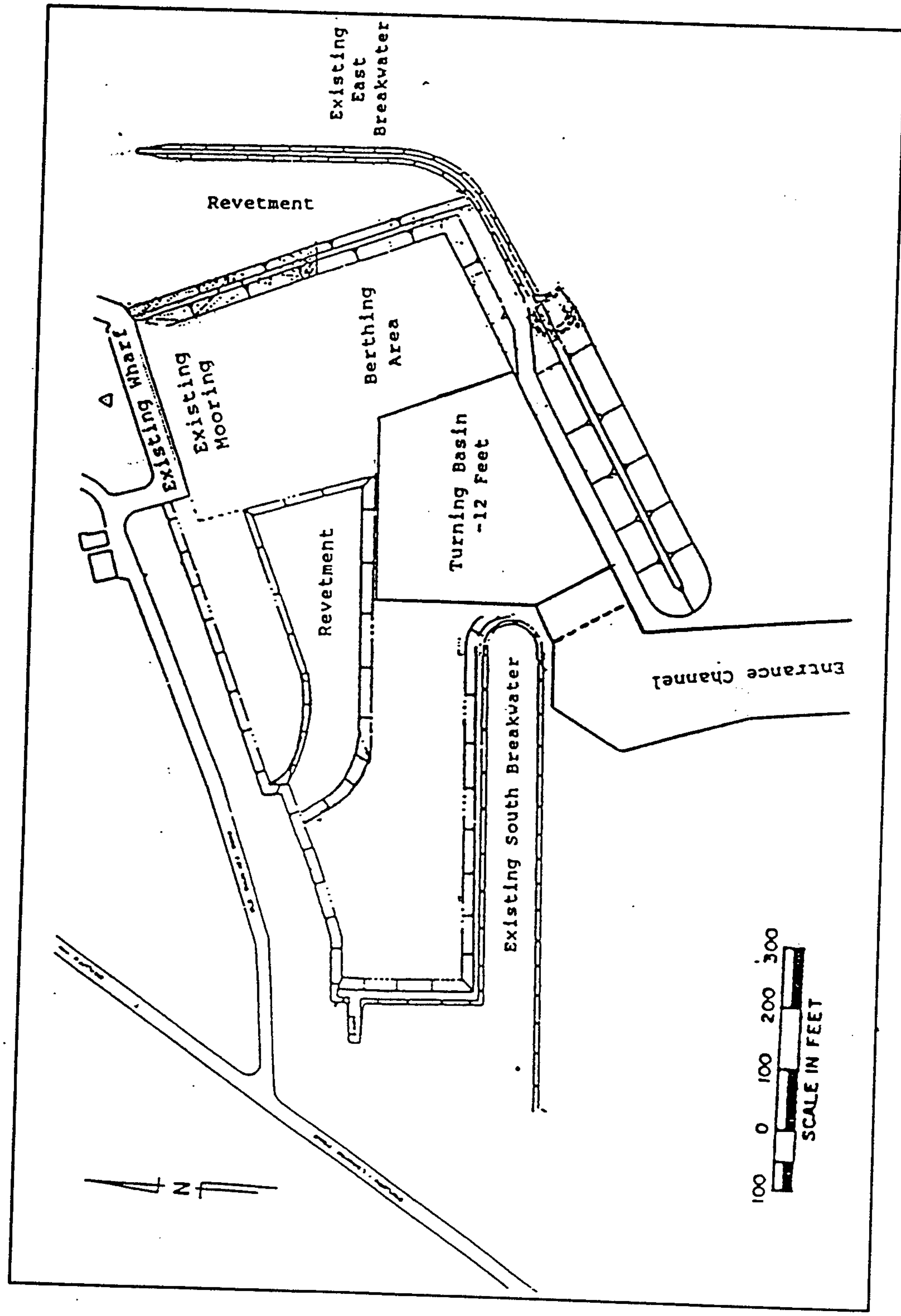

Figure 4. Proposed Plan 2

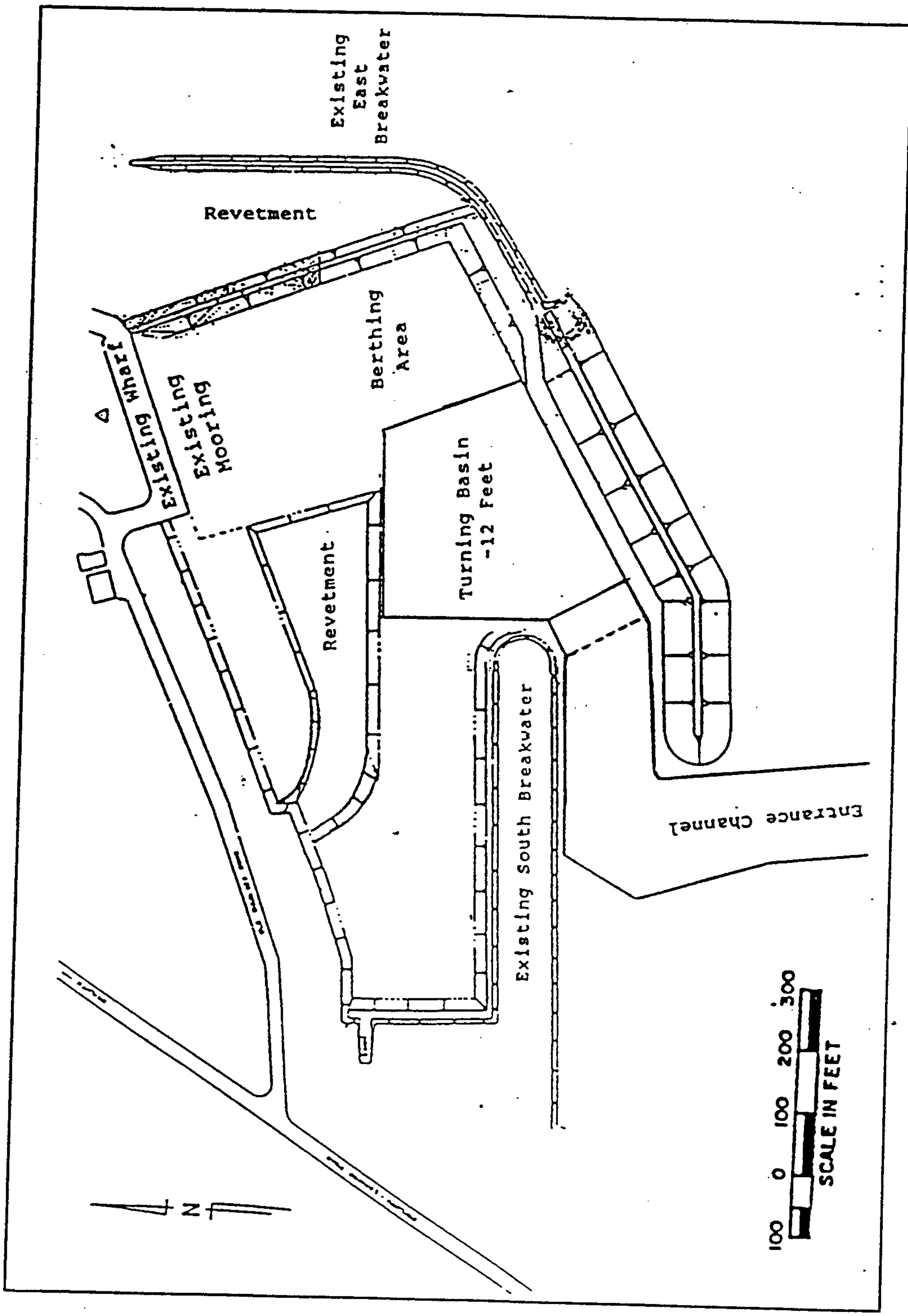

Figure 5. Proposed Plan 3

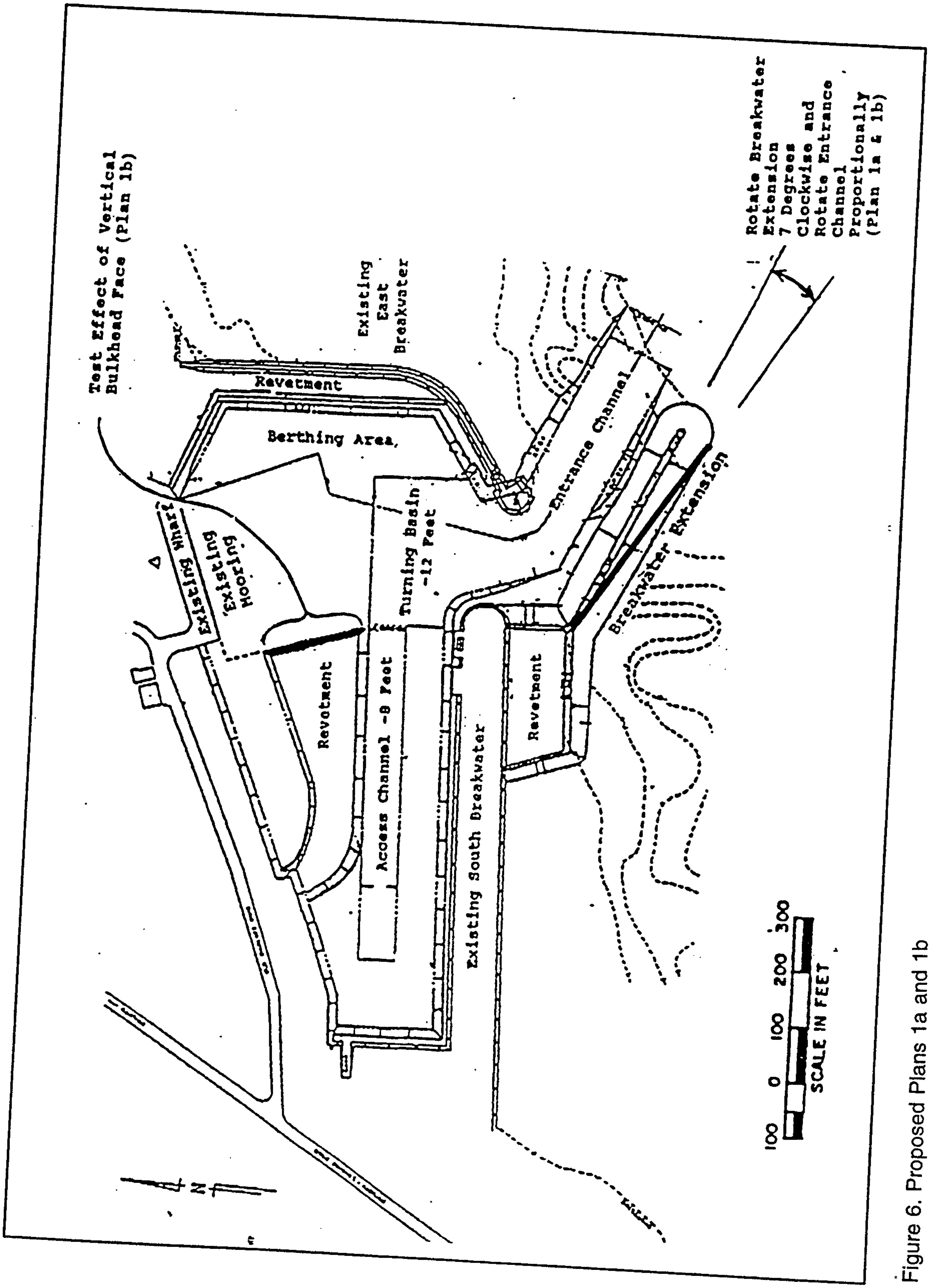

Figure 6. Proposed Plans 1a and 1b

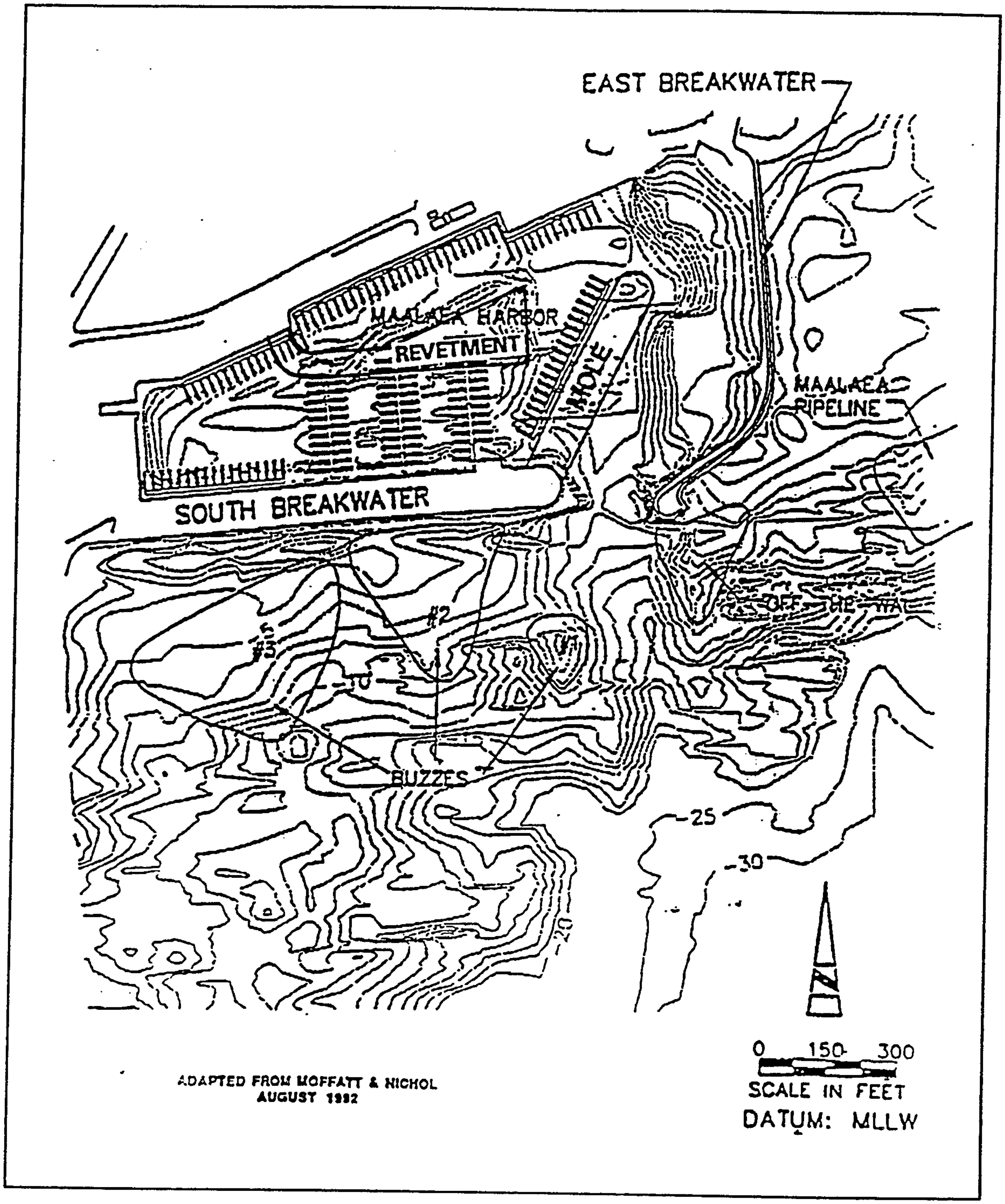

Figure 7. Proposed Plans 6 and 6a

b. A 610-ft-long entrance channel, varying in width from 150 to 200 ft, and varying in depth from 12 to 15 ft (not shown in Figure 7).

c. A 570-ft-long interior revetment varying in width from 50 to 170 ft.

Plan 6a is a structurally identical variation of Plan 6 in which dredging of the existing harbor area is limited to the harbor interior. Plan 6a was considered to assess the effects of sediment accumulation in the channel entrance after initial dredging to the design depth. This plan also takes into account the possibility that funding for extensive dredging and maintenance of channel areas outside of the harbor would be unavailable. In this plan the existing entrance channel is dredged to a uniform depth of 10 ft with no additional dredging exterior to the harbor entrance.

Study objectives of the Headquarters, U.S. Army Corps of Engineers (HQUSACE) and POD were to test the proposed harbor design improvements against the criteria that wind wave and swell wave heights not exceed 1 ft in berthing areas and 2 ft in the entrance and access channels and turning basin more than approximately 10 percent of the time per year. Another objective was to assess the potential for harbor oscillations in all plans relative to the existing harbor. To accomplish these objectives, the numerical harbor wave response model HARBD (Chen and Houston 1987) developed at USAEWES was used to test the existing harbor configuration and proposed plans.

Modeling Approach

Both numerical and physical modeling were originally considered for the study of alternative modifications to Maalaea small boat harbor. As discussed by Lillycrop et al. (1993), the numerical modeling approach was chosen to assess the variety of proposed alternatives. Assumptions inherent in the numerical modeling approach are as follows:

a. No wave transmission or overtopping of structures.

b. Structure crest elevations will not be tested or optimized.

c. No wave-wave or wave-current interaction.

d. No wave breaking effects.

e. Diffraction around the structure ends is represented by diffraction around a blunt vertical wall with specified reflection coefficient.

f. Energy losses at constricted entrances are not explicitly included.

Within the limits of the assumptions, the numerical modeling approach can be expected to give a reasonable assessment of the proposed plans.

The procedures originally used to develop incident wind wave and swell information for the harbor response model are described by Lillycrop et al.

(1993). The HARBD model and finite element grids used are presented in Chapter 2. The updated wind wave and swell results, including a discussion of the NDBC buoy data used as the deepwater wave climate in this study, are

given in Chapter 3. Harbor oscillation results for all plans, including the Existing Plan, are given in Chapter 4. Evaluation of proposed improvement plans based upon navigational concerns is given in Chapter 5. Conclusions are summarized in Chapter 6.

2 Numerical Model

Model Formulation

The numerical wave model HARBD is a steady state hybrid element model used in the calculation of linear wave response in harbors of varying size and depth (Chen 1986, Chen and Houston 1987, Lillycrop and Thompson 1996). Originally developed for use with long period waves (Chen and Mei 1974), HARBD has since been adapted to include capabilities for modeling wind waves and swell (Houston 1981), bottom friction, and partially reflective boundaries (Chen 1986). The model is based on a linearized mild slope equation. An overview of the model and its applications is given by Thompson and Hadley (1995).

The HARBD model has been shown to perform satisfactorily in comparison to analytic solutions and laboratory data for a variety of wind wave and swell cases (Houston 1981, Crawford and Chen 1988, Thompson et al. 1996) and long wave cases (Chen 1986, Chen and Houston 1987, Houston 1981, Thompson et al. 1993). As a result it has been used with confidence in both long wave and short wave studies. Long wave studies have included harbor oscillations (Thompson et al. 1997, Smith et al. 1997, Thompson et al. 1996b, Thompson and Hadley 1994b, Briggs et al. 1994, Briggs et al. 1992, Mesa 1992, Sargent 1989, Weishar and Aubrey 1986, Houston 1976) and tsunamis (Farrar and Houston 1982, Houston and Garcia 1978, Houston 1978). Wind wave and swell studies include Thompson et al. (1996b), Thompson and Hadley (1994a, 1994b), Lillycrop et al. (1990), Lillycrop and Boc (1992), Lillycrop et al. (1990), Kaihatu et al. (1989), Farrar and Chen (1987), Clausner and Abel (1986), and Bottin et al. (1985).

The HARBD model covers in detail a domain including the harbor and a portion of the adjacent nearshore area (Figure 8). This domain is bounded by a 180-deg semicircle in the water region seaward of the harbor entrance (∂A in Figure 8) and the land-water interface along the shoreline and harbor (∂C in Figure 8). The region defined by these boundaries is denoted Region A. If possible, the semicircle radius should be at least twice the wavelength of the longest incident wave to be modeled (using a typical water depth within the semicircle). Also, the semicircle should encompass any complex offshore bathymetry which strongly influences waves entering the harbor. In general,

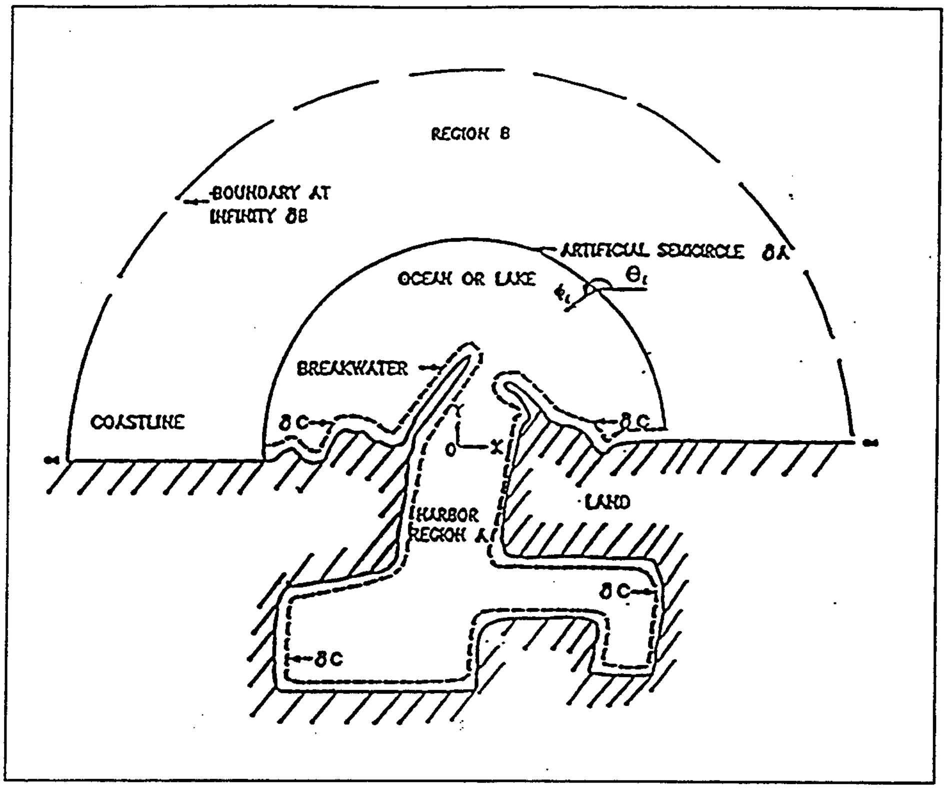

Figure 8. Representation of HARBD domain

the semicircle should be as large as practical constraints on grid size and resolution will allow.

The area outside the semicircle is treated as a semi-infinite region which extends from a straight coastline seaward to infinity (Region *B*). This region is assumed to have a constant water depth and no bottom friction.

Assuming linear, regular waves propagating over mild slope in arbitrary water depth, Chen (1986) derived the governing equation as

$$\nabla \cdot (\lambda \, c \, c_g \, \nabla\phi) + \frac{\omega^2 \, c_g}{c} \phi = 0 \tag{1}$$

where ∇ = horizontal gradient operator
λ = complex bottom friction factor
c = wave phase speed
c_g = wave group speed
ϕ = velocity potential
ω = angular frequency

This equation is identical to Berkhoff's (1972) equation except for addition of the bottom friction factor λ. The factor λ, which is a complex number with magnitude greater than zero and less than or equal to one, is specified as

$$\lambda = \frac{1}{1 + \frac{i\beta a_i}{d \sinh \kappa d} e^{i\gamma}} \qquad (2)$$

where $i = (-1)^{1/2}$
β = dimensionless bottom friction coefficient that can vary in space
a_i = incident wave amplitude
d = water depth
κ = wave number
γ = phase shift between stress and flow velocity

The bottom friction factor is a factor tending to reduce local velocities proportionately through the relationships

$$u = \lambda \frac{\partial \phi}{\partial x}$$
$$v = \lambda \frac{\partial \phi}{\partial y} \qquad (3)$$

where u,v = local horizontal velocity components
x,y = horizontal coordinates

Boundary conditions are specified in Regions A and B. At the solid boundary ∂C, a reflection/absorption boundary condition is used similar to the impedance condition in acoustics. The condition is specified as

$$\frac{\partial \phi}{\partial n} - \alpha \phi = 0 \qquad (4)$$

with

$$\alpha = i\kappa \frac{1 - K_r}{1 + K_r} \qquad (5)$$

where n = unit normal vector directed into the solid region
K_r = reflection coefficient of the boundary

Values of K_r for wind waves and swell are normally chosen based on the boundary material and shape. General guidelines for K_r can be assembled from laboratory and field data (Thompson et al. 1996). In wind wave and swell studies, K_r is generally chosen to be consistent with this guidance. Effects such as slope, permeability, relative depth, wave period, breaking, and overtopping can be considered in selecting values within these fairly wide ranges. For long wave studies, K_r is generally set equal to 1.0, representing full reflection.

The second boundary condition is imposed in the far region (Region B) at infinity. It requires that the scattered wave, defined as the difference between the total wave and incident wave, behave as a classical outgoing wave at infinity. This radiation condition may be expressed as

$$\lim_{r \to \infty} \sqrt{r} \left(\frac{\partial}{\partial r} - i\kappa \right) \phi^s = 0 \tag{6}$$

where r = radial polar coordinate
ϕ^s = velocity potential of the scattered wave

The complete boundary value problem is specified by Equations 1, 4, and 6. A hybrid element method is employed to solve the boundary value problem. A conventional finite element grid is developed and solved in Region A. The triangular elements allow detailed representation of harbor features and bathymetry within Region A. An analytical solution with unknown coefficients in a Hankel function series is used to describe Region B. For a given grid, short wave period tests (relatively large values of κ) require more terms than long period tests to adequately represent the series. A variational principle with a proper functional is established such that matching conditions are satisfied along ∂A. Details are given by Chen (1986) and Lillycrop and Thompson (1996).

Experience with the model has indicated that the element size Δx and local wavelength L should be related by

$$\Delta x \leq \frac{L}{6} \tag{7}$$

Typically, harbor domains include some shallow areas in which many elements would be needed to satisfy the constraint in Equation 7. In practice, Equation 7 is at least satisfied in the harbor channel and basin depths. If additional elements can be accommodated, it is generally preferred to extend the semicircle further seaward rather than to greatly refine shallow harbor regions.

Input information for HARBD must be carefully assembled. In addition to developing the finite element grid to suit HARBD requirements, a number of parameters must be specified. Critical input parameters and ranges of typical values are summarized in Table 1.

The principal output information available from HARBD consists of amplification factor and phase at each node. These are defined as

Table 1
Critical HARBD Input Parameters and Ranges of Typical Values

Parameter	Where Specified	Typical Values	
		Short Waves	Long Waves
Bottom friction, β	Every element	0.0	0.0-0.1
Boundary reflection, K_r	Every element on solid boundary	0.0 - 1.0	1.0
Coastline reflection, $K_{r,coast}$	Single value	1.0	1.0
Depth in infinite region, d_{far}	Single value	Between avg. & max. on semicircle	
Number of terms in Hankel function series	Single value	8 - 100*	8

* The number of terms needed increases as wave period decreases.

$$A_{amp} = \left|\frac{a}{a_i}\right| = \left|\frac{H}{H_i}\right| = |\phi|$$

$$\theta = \tan^{-1}\left[\frac{Im\ \{\phi\}}{Re\ \{\phi\}}\right] \tag{8}$$

where A_{amp} = amplification factor
a, a_i = local and incident wave amplitudes
H, H_i = local and incident wave heights
θ = phase relative to the incident wave
$Im\{\phi\}$ = imaginary part of ϕ
$Re\{\phi\}$ = real part of ϕ

Amplification factors are easily interpreted. Phases are helpful in viewing wind wave and swell propagation characteristics and in interpreting standing wave patterns. In long wave applications, phases prove useful for determining relative phase differences within the harbor, interpreting harbor oscillation patterns, and identifying potentially troublesome nodal areas.

Spectral Adaptation

HARBD computes harbor response to specified wave period and direction combinations. However the model is often used to approximate irregular wind wave and swell behavior, as in physical model tests with irregular waves and all field cases. More realistic numerical model simulations can be obtained by linearly combining HARBD results from a range of regular wave frequencies

in the irregular wave spectrum. Regular wave results are weighted to properly represent the desired spectral distribution of energy. The concept of linear superposition of weighted regular wave results can also be extended to include directional spreading in the spectrum to be simulated.

Spectral adaptation of the HARBD model is done as a post-processing step using the standard, regular wave output from the model. For a given incident wave direction, HARBD is run for a number of wave periods spread between the shortest period satisfying the grid resolution constraint of Equation 7 and the longest swell period of interest.

Spectral post-processing is based on the assumption that a consistent spectral form can be applied at every node. This major assumption provides the basis for a workable, reasonable spectral weighting which improves on the traditional regular wave approach. The JONSWAP spectral form was chosen (Hasselmann et al. 1973). The JONSWAP spectrum is specified as (U.S. Army Corps of Engineers 1989)

$$S(f_i) = \frac{\alpha\, g^2}{(2\pi)^4 f_i^5}\, e^a\, \gamma^b \tag{9}$$

where $S(f_i)$ = spectral energy density at frequency f_i

The parameters a and b are given by the following relationships

$$a = \frac{-1.25}{f_i\, T_p^4}$$

$$b = e^{\frac{-1}{2\sigma^2}(f_i T_p - 1)^2} \tag{10}$$

$$\sigma = 0.07 \quad \text{for } f_i \le f_p$$

$$= 0.09 \quad \text{for } f_i \ge f_p$$

where T_p = peak spectral period
f_p = peak spectral frequency = $\frac{1}{T_p}$

Parameters α and γ are calculated as

$$\alpha = 157.9\ \epsilon^2$$

$$\gamma = 6614\ \epsilon^{1.59} \qquad (11)$$

$$\epsilon = \frac{H_s}{4\ L_p}$$

where H_s = significant wave height
L_p = wavelength for waves at peak frequency

The parameter ε is a significant wave steepness. The parameter γ, called the peak enhancement factor, controls the sharpness of the spectral peak.

Although the JONSWAP spectrum was developed primarily for actively growing wind waves, it can be used with appropriate choice of γ to approximate any single-peaked spectrum, including old swell which has travelled a great distance from the generation area (e.g. Goda 1985) (Table 2).

Table 2
Guidance for Choosing γ

Wave Condition	γ
Growing sea	3.3
Old swell	8-10

Spectral post-processing begins with specification of the desired H_s, T_p, and γ and the arrays of HARBD amplification factors. A refined JONSWAP spectrum is computed with 1000 points, where the f_i's in Equation 9 are

$$f_1 = 0.5*f_p\ ,\quad f_2 = 0.502*f_p\ ,\ f_3 = 0.504*f_p\ ,\ \ldots\ ,\ f_{1000} = 2.498*f_p$$

The number of wave periods computed with HARBD is always much smaller than 1000, typically less than 20. These periods, converted to frequency (reciprocal of period), can be used to define bands in the JONSWAP spectrum. Bands are bounded by the midpoints between HARBD computational frequencies. The highest and lowest frequency bands are assumed to be centered on the highest and lowest HARBD computational frequencies, respectively. A weighting factor for each HARBD-defined band is computed by summing values from the refined JONSWAP spectrum which fall within the band and normalizing by the total spectral energy.

$$w_k = \frac{\sum_{i=N_{k1}}^{N_{k2}} S(f_i)}{\sum_{i=1}^{1000} S(f_i)} \tag{12}$$

where w_k = weighting factor for k'th HARBD computational frequency

N_{k1} = index of lowest JONSWAP frequency, f_i, satisfying

$$f_i > \frac{f_{k-1}+f_k}{2}$$

N_{k2} = index of highest JONSWAP frequency, f_i, satisfying

$$f_i < \frac{f_k+f_{k+1}}{2}$$

f_{k-1}, f_k, f_{k+1} = (k-1)'th, k'th, and (k+1)'th HARBD computational frequencies, with $f_{k-1}<f_k<f_{k+1}$

Though not shown in the equation, the weighting factor also includes fractional energy interpolated across JONSWAP frequencies bracketing the two end points of each HARBD band.

The effective amplification factor at each node is computed as

$$(A_{amp})_{eff} = \sqrt{\sum_{k=1}^{N_T} w_k\, A^2_{amp}(f_k)} \tag{13}$$

where $(A_{amp})_{eff}$ = effective , or spectral, amplification factor at a node
$A_{amp}(f_k)$ = nodal amplification factor for HARBD computational frequency f_k
N_T = number of HARBD computational wave periods

Finite Element Grids

The finite element numerical grid depicting existing conditions at Maalaea Harbor was created previously using WES's finite element grid development software (Turner and Baptista 1993) (Figure 9). The grid covers the entire Maalaea Harbor area and extends somewhat seaward from the harbor entrance. The land boundary was digitized from a NOAA nautical chart. Grid element size is based on the criterion of 6 elements per wavelength (the minimum recommended resolution with HARBD) for a 8-sec wave in 8-ft water depth. Depths over virtually the entire embayment exceed 8 ft. For the longer period

waves, the grid gives a high degree of resolution. Grid characteristics are summarized in Table 3.

Table 3
Grid Sizes

Harbor Plan	Number of:				Length of Typical Element (ft)
	Elements	Nodes	Solid Boundary Nodes	Semicircle Boundary Nodes	
Existing	7,140	3,749	252	105	20
Plan 1	6,765	3,613	355	105	20
Plans 1a & 1b	6,810	3,636	357	105	20
Plan 2	7,882	4,184	353	132	20
Plan 3	7,911	4,215	386	132	20
Plans 6 & 6a	6,747	3,603	353	105	20

The radius of the seaward semicircle is approximately 790 ft. This is equivalent to 5.7 and 2.1 wavelengths for the shortest and longest short wave periods considered, assuming a representative water depth of 10 ft. The semicircle size and location were chosen to include all breakwaters and moles and significant bathymetry south of the harbor entrance. The semicircle extends sufficiently far seaward to cover the most important nearshore bathymetry.

Bathymetric data, obtained from NOAA hydrographic charts and POD bathymetric survey data taken in 1989, were unchanged from previous studies. Depths were transferred onto the finite element grid using the USAEWES grid software package.

Reflection coefficients, K_r, are needed for all solid boundaries. For the short wave tests, K_r values were estimated from existing Corps of Engineers guidance, photos, and past experience. The solid boundary of the existing harbor was divided into seven zones and a reflection coefficient was estimated for each zone (Figure 10). Reflection coefficients ranged from 0.0 for open water areas east of the harbor to 1.0 at the wharf face along the northern portion of the harbor. Other parameter values used in the numerical model are summarized in Table 4.

Different parameters are used for long wave tests. Reflection coefficients were set to 1.0 for all boundaries, since long waves generally reflect very well from coastal boundaries. Long waves are more affected by bottom friction than short waves, so a value of β greater than zero is appropriate. The value of β is best determined by calibration with field data. A value of β=0.032 was determined for Kahului Harbor (Thompson et al. 1996b). In this case, to be

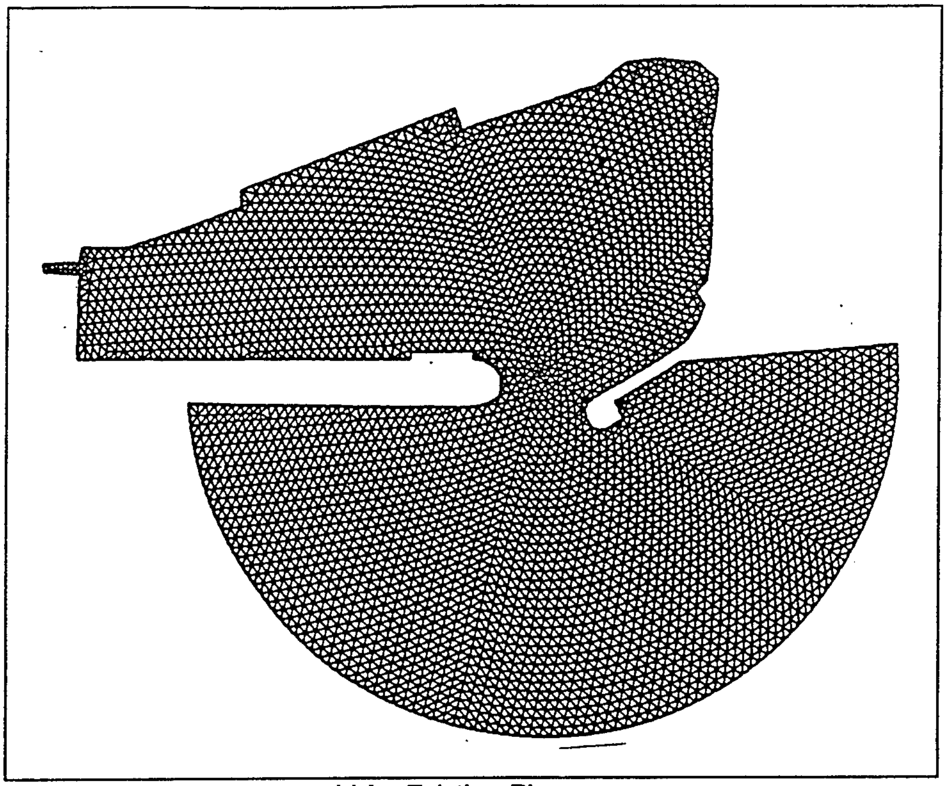

Figure 9. Finite element grid for Existing Plan

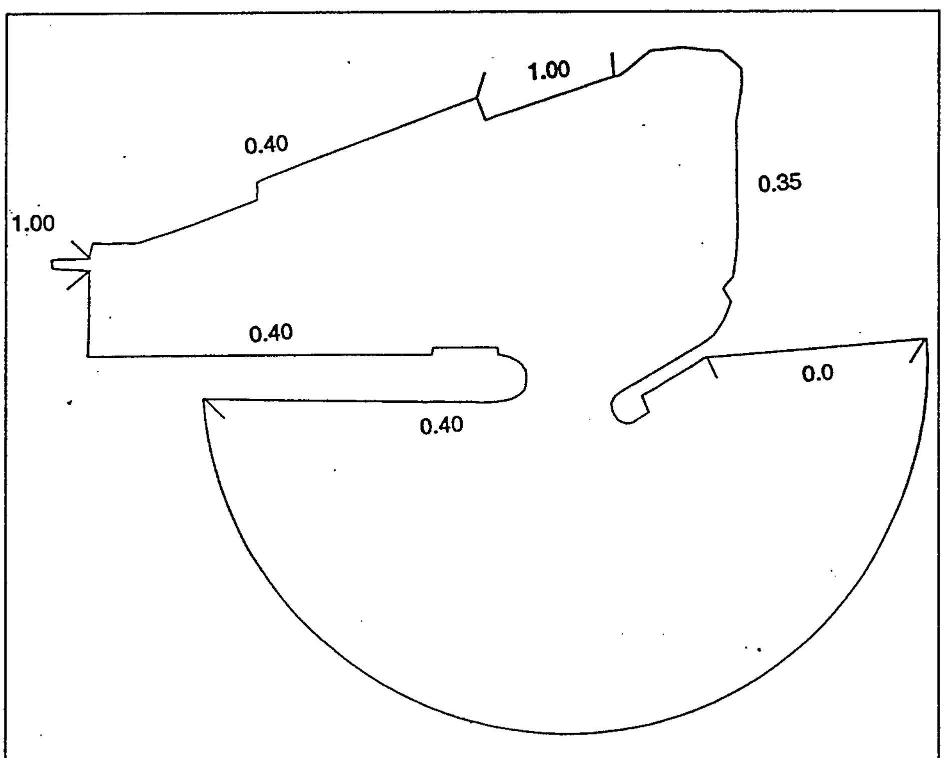

Figure 10. Boundary reflection coefficients for Existing Plan

consistent with long wave runs previously conducted for the Existing Plan and Plan 6 (Thompson and Hadley 1994b) and because an accurate value for β is not critical to the objectives of the study, a default value of β=0.0 was used. This and other parameters are summarized in Table 4.

In addition to existing conditions, seven harbor modification plans were specified for evaluation, as discussed in Chapter 1. Harbor grids were generated previously to represent each alternative configuration. Grid characteristics for each configuration are included in Table 3. Short wave reflection coefficients were modified as appropriate for each plan. General guidelines were K_r=0.40 to K_r=0.50 along moles and K_r=0.25 to K_r=0.35 along breakwater extensions.

Table 4
Parameter Values Used in HARBD

Parameter	Value	
	Short Waves	Long Waves
Bottom friction, β	0.0	0.0
Coastline reflection, $K_{r,coast}$	1.0	1.0
Depth in infinite region, d_{far}	25 ft	25 ft

3 Harbor Response to Wind Waves and Swell

Percent occurrence statistics for wind wave and swell climate in Maalaea Harbor were estimated based on deepwater wave climate percent occurrence tables. For this study, the deepwater wave climate was taken from 12 months of data (Dec 94 through Nov 95) from National Data Buoy Center buoy 51027, located approximately 25 miles southwest from the island of Lanai (Appendix A). The buoy had an open exposure to wave directions of importance to Maalaea Harbor. Only those deepwater directions likely to impact Maalaea Harbor were considered. Percent occurrences for these directions were taken directly from the buoy climate, assuming that Maalaea Harbor would be calm for cases when the buoy recorded wave directions headed away from the harbor. The buoy is a much more reliable source of deepwater wave information than was available when earlier studies of Maalaea Harbor were conducted. This change contributes significantly to the reliability of study results.

To establish wave climate incident to Maalaea harbor, a total of 187 deepwater wave height, period, and direction combinations were input to the SHALWV model (Lillycrop et al. 1993). The SHALWV grid extended beyond the island of Kahoolawe. It allowed estimates of sheltering and shallow water effects on waves between the deepwater, open ocean south of Kahoolawe and the Maalaea harbor area. To determine wave heights in Maalaea harbor, SHALWV wave heights near the harbor (in the vicinity of the seaward boundary of the HARBD grid) were multiplied with the HARBD amplification factors corresponding to each deepwater condition. The 187 wave height, period, and direction combinations were tested. All simulations were run on the WES CRAY Y-MP and SGI PCA1 supercomputing facilities.

Output "basins" were selected for each plan to determine wave response throughout the harbor. A basin is a small cluster of elements over which the HARBD response is averaged to give a more representative output. The number of basins for each plan varied between 16 and 24. The locations, selected by WES and POD, are shown for the Existing Plan in Figure 11 and in Appendix B for all remaining plans. Since the wave height criteria which must be satisfied differ for channel areas and berthing areas, output basins for each plan are designated by area (Table 5).

The percent occurrences of wave heights exceeding 1 ft in the berthing areas and 2 ft in the entrance and access channels and turning basin were calculated for all plans. The procedure is based upon the same principles employed by Lillycrop et al. (1993).

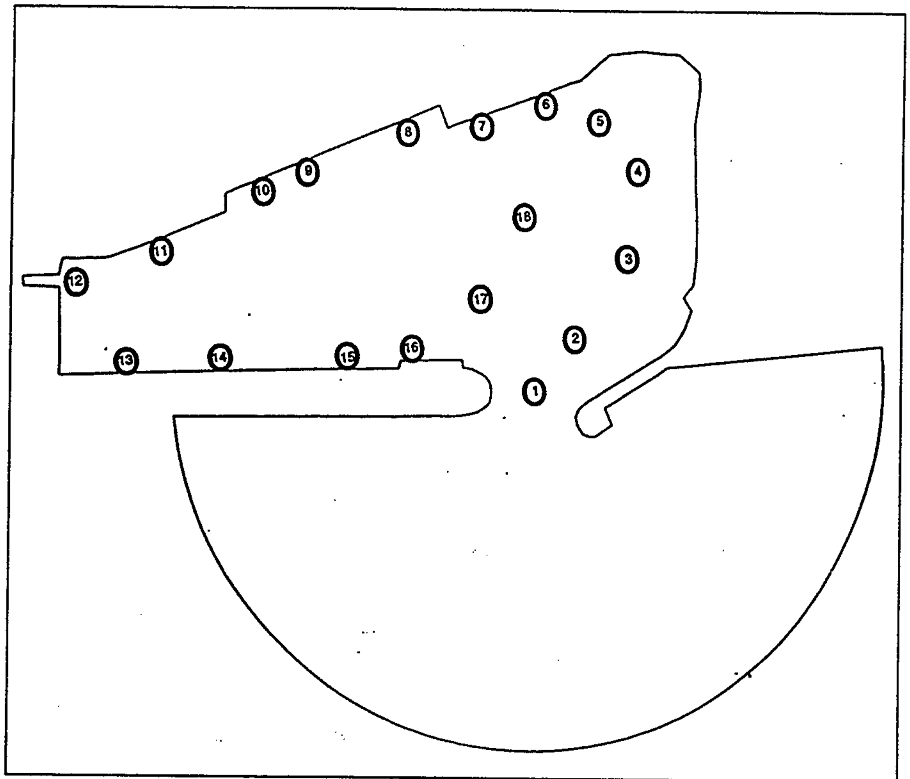

Figure 11. Output basin locations for Existing Plan

Percent occurrence of wave heights exceeding 2 ft in entrance channels is related to the amount of exposure to incident waves. Basins with full exposure to incident waves typically have a higher percentage of exceedence than basins located in more protected channel areas. For Maalaea Harbor wave studies, the most seaward basin in the entrance channel of each harbor plan was placed at or near the entrance constriction point where vessels have minimum maneuvering space between harbor structures while being subjected to ocean wave forces.

Table 5
Designation of Output Basin Areas

Plan	Basin Numbers	
	Channel	Berthing
Existing	1-6	7-18
Plans 1, 1a, 1b	1-5	6-23
Plan 2	1-6	7-23
Plan 3	1-7	8-24
Plans 6, 6a	1-6	7-18

HARBD amplification factors from which percent occurrences were generated were obtained by first running a range of short wave conditions (Table 6) encompassing minimum and

maximum periods and directions of the full array of short wave input conditions generated from SHALWV results. Model results were then evaluated for cdirectional spectra with peak period and direction values equivalent to the original array of short wave input conditions.

Table 6
Summary of Short Wave Periods and Directions

Wave Period (sec)	Wave Direction (deg, azimuth)
8	202
9	195
11	185
13	175
15	165
17	160
20	

Tabulations of the HARBD-SHALWV wave heights initially exceeding the HQUSACE criteria for each deepwater wave direction are given in Appendix C. For the Existing Plan, Table C1 shows that the wave heights initially exceeding the maximum 1 ft criterion in berthing areas (basins 7 through 18) resulted from 9-sec waves coming from the 135-, 157.5-, 180-, 202.5-, and 225-deg directions; 11-sec waves from the 157.5-, 180-, 202.5-, and 225-deg directions; 13-sec waves from the 157.5-, 180-, 202.5-, 225-, and 247.5-deg directions; 15-sec waves from the 135-, 157.5-, 180-, 202.5-, 225-, and 247.5-deg directions, 17-sec waves from the 135-, 157.5-, 180-, and 202.5-deg directions; and 20-sec waves from the 157.5- and 180-deg directions. Predominantly, initial exceedence occurred at basin 7, along the existing wharf, with a few cases of initial exceedence at basins 17 and 18. Wave heights exceeding the 2 ft maximum criterion in the entrance channel (basins 1-6) resulted from 9-sec waves coming from the 157.5-, 180-, 202.5-. and 225-deg directions; 11-sec waves from the 157.5- and 180-deg directions; 13-sec waves from the 157.5-, 180-, 202.5-, and 225-deg directions; 15-sec waves from the 157.5-, 180-, 202.5- and 225-deg directions; 17-sec waves from the 157.5-, 180-, and 202.5-deg directions; and 20-sec waves from the 157.5- and 180-deg directions. These waves occurred at the harbor entrance in basin 1.

Tables C2 and C3, for Plan 1 and Plan 1a respectively, show that with a single exception, wave heights initially exceeding the maximum 1 ft criterion in berthing areas did not occur for deepwater incident wave heights of 9 ft or less. The exception is a 9 ft, 17-sec wave from the 180-deg direction which exceeds at basin 11 of Plan 1a. There was no occurrence of wave heights initially exceeding the maximum 2 ft criterion in the entrance channel for either of the plans.

Table C4 (Plan 1b) shows that the wave heights initially exceeding the maximum 1 ft criterion in berthing areas (basins 6 through 23) resulted from 13-sec waves coming from the 180-deg direction; 15-sec waves from the 157.5- and 180-deg directions; 17-sec waves from the 157.5- and 180-deg directions; and 20-sec waves from the 157.5- and 180-deg directions. There were no instances of exceedence of the 2 ft criterion for deepwater incident wave heights less than 9 ft for Plan 1b.

Wave conditions initially exceeding the maximum 1 ft criterion in berthing areas for Plan 2 (Table C5) include 9-sec waves coming from the 135-, 157.5-, 180-, 202.5-, and 225-deg directions; 11-sec waves from the 157.5- and 180-deg directions; 13-sec waves from the 157.5-, 180- and 202.5-deg directions; 15-sec waves from the 157.5-, 180-, and 202.5-deg directions; 17-sec waves from the 157.5-, 180-, and 202.5-deg directions; and 20-sec waves from the 157.5-and 180-deg directions. Exceedence occurred primarily at basins 7 and 8, near the north end of the east breakwater, and basin 23, near the tip of the west mole. Wave heights exceeding the 2 ft maximum criterion in the entrance channel resulted from 9-sec waves from the 135-, 157.5-, 180-, 202.5-, and 225-deg directions; 11-sec waves from the 157.5- and 180-deg directions; 13-sec waves from the 157.5-, 180-, 202.5-, and 225-deg directions; 15-sec waves from the 157.5-, 180-, 202.5- and 225-deg directions; 17-sec waves from the 157.5-, 180- and 202.5-deg directions; and 20-sec waves from the 157.5- and 180-deg directions. These waves occurred at the harbor entrance in basin 1.

As shown in Table C6, none of the deepwater wave conditions resulted in wave heights exceeding the maximum 1- and 2-ft criteria for Plan 3. However, the percent occurrence of wave heights greater than 9 ft was included in the tabulations for this plan.

For Plan 6 and Plan 6a (Tables C7 and C8), there was no exceedence of the maximum 1 ft criterion in the berthing areas for either plan. Wave heights exceeding the 2 ft criterion in the entrance channel resulted from 9-sec waves from the 157.5-, and 180-deg directions; 11-sec waves from the 180-deg direction; 13-sec waves from the 157.5- and 180-deg directions; 15-sec waves from the 157.5-, 180-, 202.5- and 225-deg directions; 17-sec waves from the 157.5-, 180-, and 202.5-deg directions; and 20-sec waves from the 157.5- and 180-deg directions in both plans. Plan 6a also experienced exceedence of the 2 ft criterion for the additional conditions of 9- and 13-sec waves from the 202.5-deg direction and 13-sec waves from the 225-deg direction. Initial exceedence occurred at basin 1 in all cases.

The percent occurrence of wave heights exceeding the maximum 1-ft and 2-ft criteria for each plan was calculated using the percent occurrence tables of deepwater conditions and HARBD-SHALWV wave height results. These results are given in Appendix D. Although wave breaking was not taken into account in the tables, higher wave heights may break over the reef, thus reducing wave heights in the harbor. In evaluating the percent occurrence results, it is apparent that waves approaching from the west to southwest (270.0 to 247.5 deg) directions are insignificant in comparison to waves approaching from the southwest to southeast (225.0 to 135.0 deg) directions.

The percentage of wave heights exceeding the maximum 1-ft and 2-ft criteria for the Existing and Plans 1, 1a, 1b, 2, 3, 6, and 6a are summarized in Table 7 along with the HQUSACE criteria. These values are somewhat conservative since they represent basins with the largest wave heights occurring in the harbor for each deepwater wave condition.

Table 7
Summary of Percent Occurrence of Wave Heights

Location	Percent of Time Criterion is Exceeded								
	USACE Crit.	Exist. Plan	Plan 1	Plan 1a	Plan 1b	Plan 2	Plan 3	Plan 6	Plan 6a
Berthing areas (1 ft crit.)	< 10	32.8	0.6	0.8	1.6	10.8	0.6	0.6	0.6
Entrance Channel (2 ft crit.)	< 10	15.4	0.6	0.6	0.6	18.1	0.6	8.8	13.5

The Existing Plan allows the 1 ft wave height criterion in the berthing areas to be exceeded 32.8 percent of the time per year. This violates the HQUSACE standard that wave heights exceed 1 ft in these areas no more than 10.0 percent of the time per year. The entrance channel in the Existing Plan shows an exceedence of 15.4 percent of the time per year of the 2 ft wave height criterion which also exceeds the HQUSACE standard.

Plans 1 and 1a, which include structural modification to the east, and Plan 3, which includes structural modification to the west, allow exceedence of the 1- and 2-ft criteria less than 1 percent of the time per year. This falls well below HQUSACE guidelines for providing adequate protection in the berthing and channel areas. Plan 1b also falls below the guidance, exceeding the criteria less than 2 percent of the time. Plan 2 shows exceedence of the 1 ft and 2 ft criterion 10.8 and 18.1 percent of the time per year, respectively, which exceeds HQUSACE guidelines. Plans 6 and 6a both fall below the HQUSACE guidance for berthing areas with an exceedence of the 1 ft criteria less than 1 percent of the time. Plan 6a, however, exceeds the 2 ft wave height criteria 13.5 percent of the time per year while Plan 6 is marginally acceptable with an exceedence of 8.8 percent of the time per year.

4 Harbor Oscillations

The HARBD numerical model was run for all plans, including the Existing Plan, to investigate harbor response to wave periods characteristic of harbor oscillations. These tests were included because the "surge" problem reported in the existing harbor may arise in part from a resonant response to long period wave energy impacting the harbor. Harbor oscillations were not considered in the earlier study by Lillycrop et al. (1993), but were considered by Thompson and Hadley (1994b) for the Existing Plan and Plan 6. Runs for both of these plans were repeated in the present study in order to incorporate changes in the modeling technology. Current results for the Existing Plan differ significantly from those obtained by Thompson and Hadley (1994b). Differences in results for Plan 6 were negligible.

Incident long wave conditions considered are given in Table 8. A fine resolution in wave frequency was used over the full range of possible resonant conditions to ensure that all important peaks were identified. Only one approach direction is included, since past studies have indicated that harbor response is relatively insensitive to incident long wave direction. This direction represents a wave directly approaching the harbor entrance from deep water.

Table 8
Summary of Incident Long Wave Conditions

Wave Period (sec)	Wave Direction (deg, azimuth)
20.00	180
20.08	
20.16	
...[1]	
780.00	

[1] Frequency increments are 0.0002 Hz for periods of 20-100 sec and 0.00007 Hz for periods of 100-780 sec

Amplification factors for all improvement plans compared to the existing harbor plan are shown for selected corner basins in Appendix E. It is important to note that although basin numbers for individual plans may differ from those of the existing harbor plan, locations of the basins are comparable. Coincident basin locations allow for a more straightforward comparison of oscillation characteristics of harbor configurations.

Figures E1 through E6 show amplifications at the west end of the harbor basin nearest the small boat ramp. In general, Plans 1, 1a, and 1b, show significantly higher peak amplifications over the Existing Plan at this location for the

range of frequencies from 0.01 to 0.05 Hz, particularly at the higher frequencies (0.02 - 0.05 Hz). Plans 2 and 3, on the contrary, show a marked decrease in peak amplifications over the Existing Plan for the same range of frequencies, with the exception of two notable peaks between 0.04 and 0.05 Hz. Plans 6 and 6a, also display higher peak amplifications over the Existing Plan for lower frequency waves (0.01 to 0.025 Hz) while showing lower peak amplifications by comparison for higher frequencies (0.025 to 0.05 Hz). Harbor oscillation patterns for resonances near 0.019 Hz and 0.025 Hz were given by Thompson and Hadley (1994b) for Plan 6.

Figures E7 through E12 show amplifications at a point located along the north boundary of the harbor basin. This point is significant relative to the Existing Plan due to the addition of a new "corner" area created by the development of the interior revetment. Plans 1, 1a, and 1b (Figures E7 and E8) show a marked decrease in the number of resonant peaks as well as significant reduction in the magnitude of amplification compared to the Existing Plan for higher frequency waves (0.035 to 0.05 Hz). For lower frequency waves (0.01 to 0.035 Hz), these plans show little difference in the number of resonant peaks but display comparable or increased magnitudes of amplification. Plans 2 and 3 (Figures E9 and E10) also show reductions in both the number of resonant peaks and the magnitude of the amplifications relative to the Existing Plan (with exceptions), but over the full range of frequencies from 0.01 to 0.05 Hz. Plans 6 and (Figures E11 and E12) give results similar to those of Plan 1, 1a, and 1b, with higher peak amplifications over the Existing Plan at lower frequencies (0.01 to 0.025 Hz) and decreased amplifications at higher frequencies. There is a single exception to this trend, a sharp but relatively small peak at approximately 0.036 Hz for Plan 6.

The new corner area may act as an antinode for a number of different resonant modes in several of the plans, as indicated by high amplification factor peaks. The strong response could make this region less desirable for berthing facilities. However, amplification factors shown in Appendix E should be viewed as conservatively high for several reasons. Wave reflection coefficients at all solid boundaries were taken as 1.0. Bottom friction was neglected (β=0.0). Energy losses through a constricted entrance are not explicitly included in the HARBD model (Thompson et al. 1993). Finally, the east breakwater in each plan is represented as a solid barrier; but for harbor oscillation wave periods, significant energy may be transmitted through it.

Based on experience with field data and numerical modeling of various harbors employing nonzero bottom friction and boundary reflections varying from 1.00 at low frequencies to approximately 0.95 for higher oscillation frequencies, it is expected that lower frequency resonances, ranging from about 0.005 to 0.025 Hz, are the most significant considerations. Thus, the plan conditions, especially Plans 1, 1a, 1b, 6, and 6a, may be expected to oscillate more than the existing harbor in the semi-enclosed area north of the plan revetted interior mole. However, differences in overall strength of oscillation between the existing and plan harbors appear to be relatively small, and long wave activity in other harbor areas should be comparable to the existing harbor.

5 Navigation

Introduction

A primary objective in harbor entrance design is to provide a safe passage for boats to enter and exit the harbor, while maintaining adequate protection of the harbor interior from wave action. Engineering design guidance is available to determine a channel width and depth which will permit safe navigation. That guidance has been applied in formulating the plan alternatives for Maalaea Harbor. However, navigation guidelines concerning layout of the entrance channel and protective harbor structures are not well established. Judgement and experience must be used to insure that plan entrances will function effectively over a sufficient range of environmental conditions. This chapter addresses navigation concerns relative to the Maalaea Harbor plans.

WES Experiments

The WES has an ongoing research study of small boat response in a variety of wave environments. Preliminary results from the research study are available and they have relevance to Maalaea Harbor. Experiments were conducted in open water with the conditions given in Table 9. Other vessel lengths are being tested, but the data have not yet been analyzed.

Vessels approaching Maalaea Harbor typically experience a following wave environment (waves approaching the harbor from approximately the same direction as the vessel). The WES experiments indicate that for this situation, the vessel may be difficult to control. The most influential experimental variables were vessel speed and wavelength, though wave height was also a factor. The vessel was under control at the highest speed (8 knots) in all cases. Also, the vessel was always controllable in the presence of the shortest wavelength (0.5 L_s). At speeds less than 8 knots and wavelengths longer than 0.5 L_s, the vessel begins losing maneuverability. At vessel speeds of 4 knots or less, the vessel stops responding to the rudder, indicating a complete loss of control.

Vessel controllability, as determined from the limited number of WES experiments available, is summarized in Figure 12. In the zone of no control, the vessel is likely to be carried in the direction of wave travel. There is also

a possibility that the waves could cause the vessel to broach (turn sideways to the waves and capsize).

Table 9
Experimental Conditions for WES Small Boat Navigation Tests

Variable	Symbol	Values
Vessel length	L_s	40 ft
Vessel draft	D	5.24 ft
Vessel speed	V_s	4, 6, and 8 knots
Wavelength	L	0.5 L_s, 1.0 L_s, and 2.0 L_s
Relative water depth	d/D	1.5 and 3.8
Wave height	H	Varied from 1 ft to 5 ft

Application To Maalaea Harbor Plans

The existing Maalaea Harbor and Plans 2, 3, 6, and 6a have entrance channel orientations which would require vessels to approach the harbor from the south. Plans 1, 1a, and 1b would require an approach from the southeast. For the wave climate and local exposure at Maalaea, vessels entering Plans 2, 3, 6, and 6a would be significantly more likely to experience following waves (due to both the fairly open southern exposure and refraction near the harbor) than for Plans 1, 1a, and 1b.

Since the WES experiments show that the ratio of wavelength to vessel length is a critical factor in controllability, percent exceedence statistics of that ratio in the entrance channel were estimated. The estimates are based on deepwater percent occurrence information for wave periods and the design 15-ft water depth in the outer entrance channel. Vessel lengths of 20 ft and 120 ft were considered to cover the range of vessels using Maalaea Harbor. These results indicate that 100 percent of the wave conditions in the entrance channel would give wavelengths longer than 0.5 L_s (Figure 13).

Vessel speed entering Maalaea Harbor is restricted to limit vessel wakes. Vessel speed entering the harbor is expected to be less than 5 knots. This restricted speed coupled with the wavelength to vessel length ratios indicate that vessels entering the harbor are in jeopardy of experiencing poor or no control, especially if wave heights are big. Plans 1, 1a, 1b, and 3 would be safer in this regard, because they offer a protected section of entrance channel before vessels actually enter the harbor. With these plans, vessels could maintain a higher speed and good control until they are safely behind the outer breakwater. Plans 2, 6, and 6a appear to be the most hazardous for navigation because they require entering vessels to travel at reduced speed in a fairly exposed entrance. If a vessel were to lose control in the Plan 2 entrance, it could be thrust against the south breakwater. Similarly, if a vessel loses control in the Plan 6 or Plan 6a entrance, it could be carried against the mole paralleling the channel.

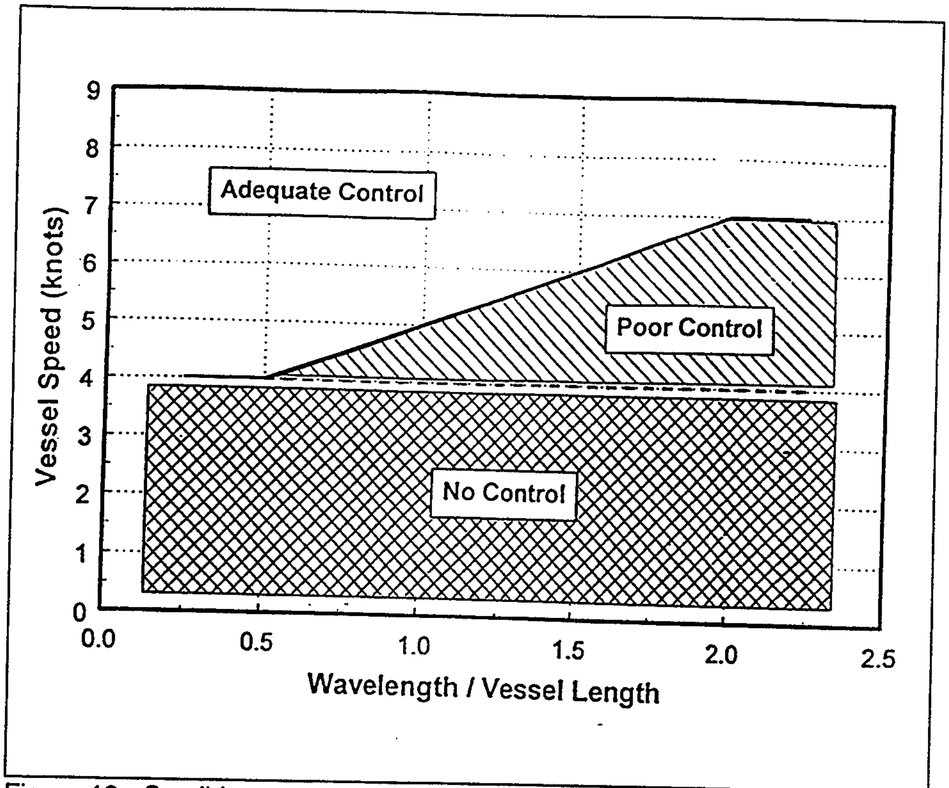

Figure 12. Small boat controllability in following waves, preliminary WES data

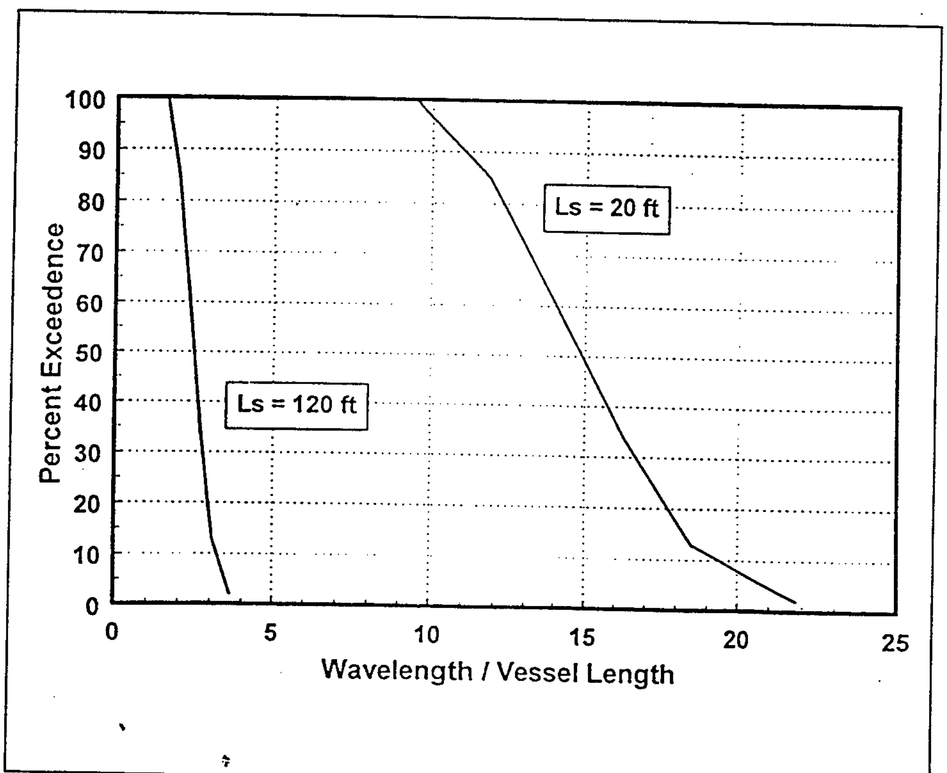

Figure 13. Wavelength percent exceedence in outer entrance channel

6 Conclusions

The numerical model studies and results described in this report should be seen in light of the following considerations:

a. Reflection coefficients were estimated as described by Lillycrop et al. (1993). Research in this area continues at WES for better guidance.

b. The following assumptions were made in the implementation of the HARBD numerical model used in this study. The model does not consider wave transmission through the breakwater, overtopping of structures, and wave breaking effects in the entrance channel; structure crest elevations were not tested or optimized; currents and nonlinear effects were neglected; and diffraction around the structure ends was represented by diffraction around a blunt vertical wall with specified reflection coefficients. If wave transmission through the breakwater and overtopping of structures did occur in the harbor, the increased energy could result in larger wave heights than predicted. The presence of wave currents and breaking would increase hazardous navigation, however wave breaking would reduce the energy in the harbor and result in lower wave heights than predicted. The primary effects which must be considered within a harbor such as Maalaea are wave refraction, diffraction, and dissipation effects for which the model has been well verified.

c. Energy losses for long period (harbor oscillation) waves passing through a constricted entrance were not explicitly modeled.

Based on the results of this study, the following conclusions were reached:

a. All of the proposed harbor plans show some degree of improvement over the Existing Plan in providing protection from incident wind waves and swell to berthing areas. All but the Existing Plan and Plan 2 satisfy the HQUSACE criterion for adequate harbor protection in these areas.

b. All of the proposed harbor plans, with the exception of Plan 2, show improvement over the Existing Plan in providing protection from incident wind waves and swell to entrance channel areas. Plans 1, 1a, 1b, and 3 appear to offer the most

protection, falling well below HQUSACE criterion for these areas. Plan 6 falls marginally below HQUSACE criterion. Plans 2 and 6a exceed the criterion significantly.

c. Navigation during high wave conditions is potentially more hazardous in Plans 2, 6, and 6a relative to other plans because they will require vessels to travel at reduced speed through a constricted entrance exposed to wind waves and swell.

d. Plans 1, 1a, 1b, 6, and 6a may be expected to experience stronger oscillations than the existing harbor, particularly at lower frequencies. The increase is due to the addition of the internal mole and breakwater structures, which can potentially lead to a significant increase in the amplitude of harbor oscillations by creating more confined corners (which can act as antinodes) in desired berthing areas. Differences in the overall strength of oscillation between the existing and plan harbors at higher frequencies appear to be small.

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Appendix A
Deepwater Wave Climate

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 135.0 DEGREES AZIMUTH

NO. OF CASES: 347
% OF TOTAL: 4.15

HEIGHT(FEET)	PERIOD(SECONDS) <8.0	8.0-9.9	10.0-11.9	12.0-13.9	14.0-15.9	16.0-17.9	18.0-21.9	22.0-LONGER	TOTAL
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		518	207						725
3.00 - 3.99	656	2037	242	35	35	35			3040
4.00 - 4.99	1899	2485	311	35	69				4799
5.00 - 5.99	656	828	345	69	35				1933
6.00 - 6.99	345	552	173						1070
7.00 - 7.99	35	207							242
8.00 - 8.99	35	69			35	35			174
9.00 - GREATER									0
TOTAL	3626	6696	1278	139	174	70	0	0	

MEAN HS(FT) = 4.6 LARGEST HS(FT) = 8.9 MEAN TP(SEC) = 8.5 NO. OF CASES = 347.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 157.5 DEGREES AZIMUTH

NO. OF CASES: 465
% OF TOTAL: 5.56

HEIGHT(FEET)	PERIOD(SECONDS) <8.0	8.0-9.9	10.0-11.9	12.0-13.9	14.0-15.9	16.0-17.9	18.0-21.9	22.0-LONGER	TOTAL
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		242	449	173	35				899
3.00 - 3.99	587	1691	621	794	759	69			4521
4.00 - 4.99	967	1450	483	1036	932	104	35		5007
5.00 - 5.99	621	587	69	311	725	311	104		2728
6.00 - 6.99	173	173		35	414	552	69		1416
7.00 - 7.99					104	1001	35		1140
8.00 - 8.99						276	35		311
9.00 - GREATER						35			35
TOTAL	2348	4143	1622	2349	2969	2348	278	0	

MEAN HS(FT) = 4.8 LARGEST HS(FT) = 9.2 MEAN TP(SEC) = 11.6 NO. OF CASES = 465.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 180.0 DEGREES AZIMUTH

NO. OF CASES: 852
% OF TOTAL: 10.19

HEIGHT(FEET)	PERIOD(SECONDS)								TOTAL
	<8.0	8.0- 9.9	10.0- 11.9	12.0- 13.9	14.0- 15.9	16.0- 17.9	18.0- 21.9	22.0- LONGER	
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		173	621	1657	1588	173			4212
3.00 - 3.99	276	380	897	2761	3038	345	35		7732
4.00 - 4.99	414	311	1208	2106	4522	828			9389
5.00 - 5.99	35	311	483	967	2623	863	69		5351
6.00 - 6.99				173	1174	518	173		2038
7.00 - 7.99				35	173	207			415
8.00 - 8.99						173	104		277
9.00 - GREATER									0
TOTAL	725	1175	3209	7699	13118	3107	381	0	

MEAN HS(FT) = 4.4 LARGEST HS(FT) = 8.6 MEAN TP(SEC) = 13.4 NO. OF CASES = 852.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 202.5 DEGREES AZIMUTH

NO. OF CASES: 337
% OF TOTAL: 4.03

HEIGHT(FEET)	PERIOD(SECONDS)								TOTAL
	<8.0	8.0- 9.9	10.0- 11.9	12.0- 13.9	14.0- 15.9	16.0- 17.9	18.0- 21.9	22.0- LONGER	
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		35	380	621	794	35			1865
3.00 - 3.99		242	414	1484	1139	207			3486
4.00 - 4.99	104	69	380	1346	1933	414	35		4281
5.00 - 5.99	69	69		173	828	207			1346
6.00 - 6.99		35		35	311	138			519
7.00 - 7.99					35				35
8.00 - 8.99					35	35			70
9.00 - GREATER					35				35
TOTAL	173	450	1174	3659	5110	1036	35	0	

MEAN HS(FT) = 4.1 LARGEST HS(FT) = 9.3 MEAN TP(SEC) = 13.3 NO. OF CASES = 337.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 225.0 DEGREES AZIMUTH

NO. OF CASES: 144
% OF TOTAL: 1.72

HEIGHT(FEET)	PERIOD(SECONDS) <8.0	8.0-9.9	10.0-11.9	12.0-13.9	14.0-15.9	16.0-17.9	18.0-21.9	22.0-LONGER	TOTAL
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		35	69	104		35			243
3.00 - 3.99		104	345	1415	380				2244
4.00 - 4.99	104			587	518	138			1347
5.00 - 5.99			69	69	276	104			518
6.00 - 6.99	35	35		104	173				347
7.00 - 7.99	35			35	138				208
8.00 - 8.99					35				35
9.00 - GREATER					35				35
TOTAL	174	174	483	2314	1555	277	0	0	

MEAN HS(FT) = 4.4 LARGEST HS(FT) = 9.8 MEAN TP(SEC) = 12.8 NO. OF CASES = 144.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 247.5 DEGREES AZIMUTH

NO. OF CASES: 208
% OF TOTAL: 2.49

HEIGHT(FEET)	PERIOD(SECONDS) <8.0	8.0-9.9	10.0-11.9	12.0-13.9	14.0-15.9	16.0-17.9	18.0-21.9	22.0-LONGER	TOTAL
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		69	104						173
3.00 - 3.99			587	1760	207	69			2623
4.00 - 4.99	276	69	345	863	828	104			2485
5.00 - 5.99	138	69	69	552	759	35			1622
6.00 - 6.99				35	104				139
7.00 - 7.99	35			35					70
8.00 - 8.99					35				35
9.00 - GREATER				35					35
TOTAL	449	207	1105	3280	1933	208	0	0	

MEAN HS(FT) = 4.4 LARGEST HS(FT) = 9.1 MEAN TP(SEC) = 12.3 NO. OF CASES = 208.

LANAI
NDBC 51027
20.45 N, 157.13 W
PERCENT OCCURRENCE(x1000) OF HEIGHT AND PERIOD BY DIRECTION
22.5 DEGREES ABOUT 270.0 DEGREES AZIMUTH

NO. OF CASES: 544
% OF TOTAL: 6.51

HEIGHT(FEET)	PERIOD(SECONDS) <8.0	8.0-9.9	10.0-11.9	12.0-13.9	14.0-15.9	16.0-17.9	18.0-21.9	22.0-LONGER	TOTAL
0.00 - 0.99									0
1.00 - 1.99									0
2.00 - 2.99		35	138	35	35	69			312
3.00 - 3.99		104	1553	1105	897	207			3866
4.00 - 4.99	207	173	1139	3279	1622	35			6455
5.00 - 5.99	207	69	483	1622	1899	138	35		4453
6.00 - 6.99	138	69	449	621	621				1898
7.00 - 7.99	35		725	173	242				1175
8.00 - 8.99			69	35	69				173
9.00 - GREATER				138	311				449
TOTAL	587	450	4556	7008	5696	449	35	0	

MEAN HS(FT) = 5.1 LARGEST HS(FT) = 10.8 MEAN TP(SEC) = 12.5 NO. OF CASES = 544.

Appendix B
Output Basin Locations

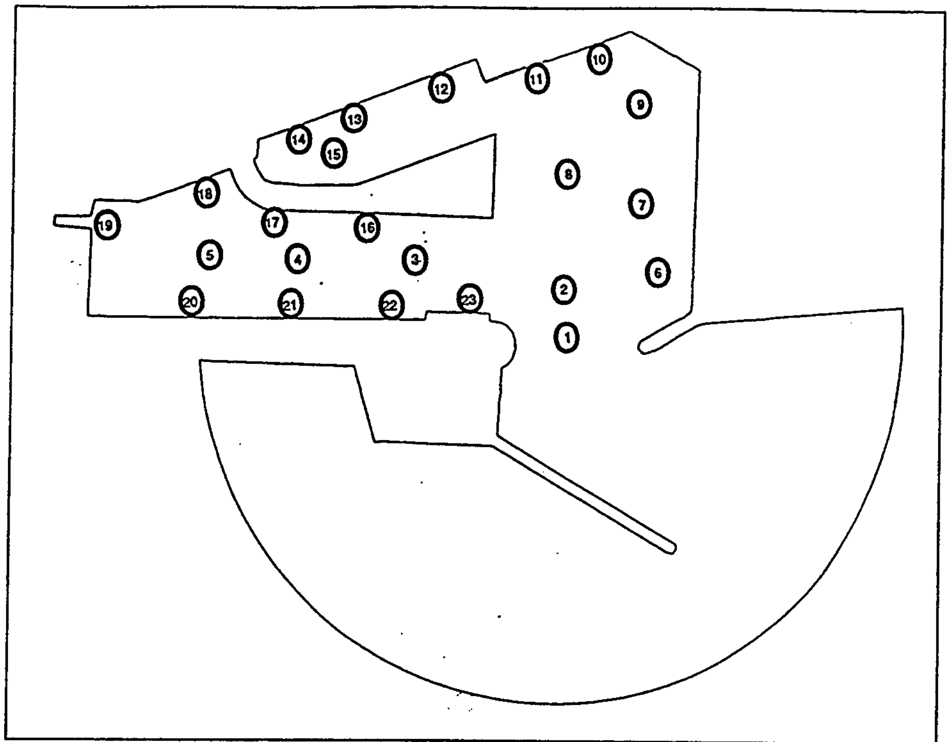

Figure B1. Output basin locations for Proposed Plans 1, 1a, and 1b

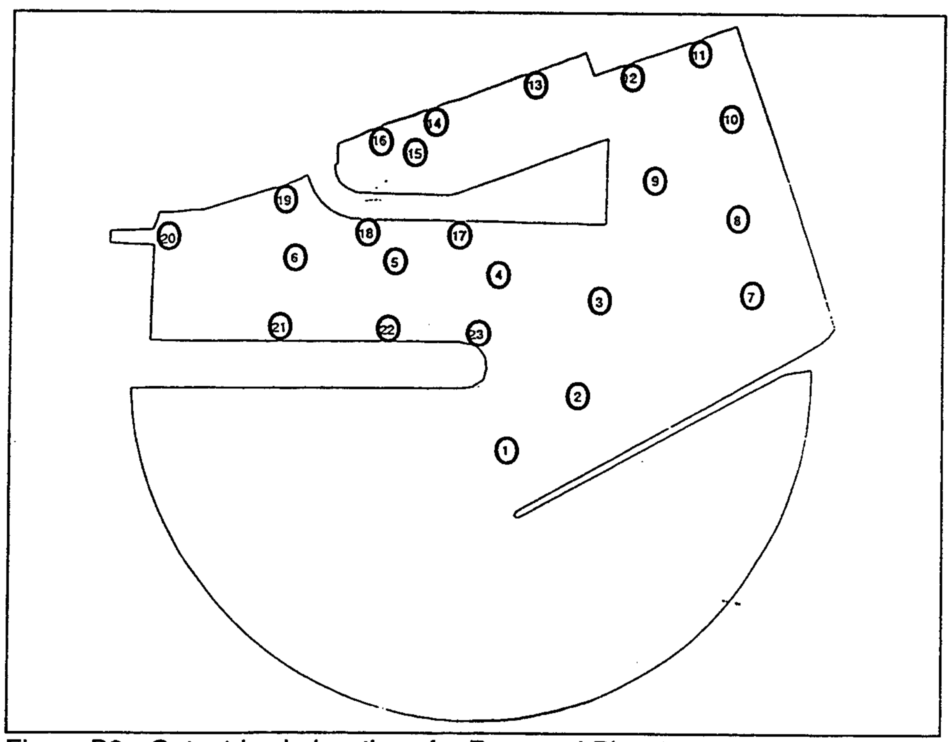

Figure B2. Output basin locations for Proposed Plan 2

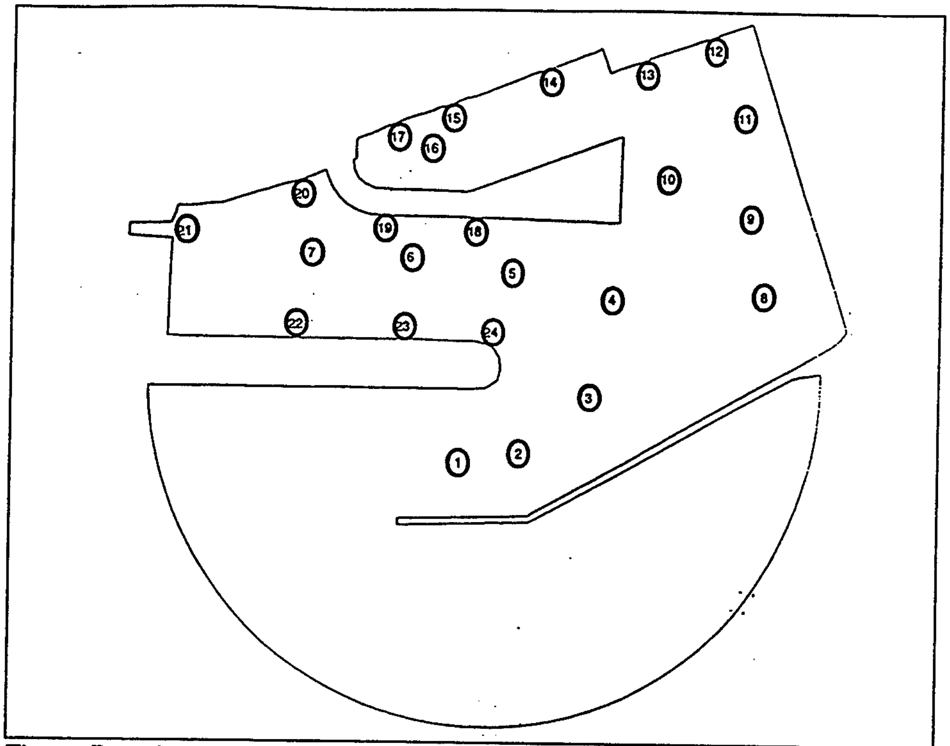

Figure B3. Output basin locations for Proposed Plan 3

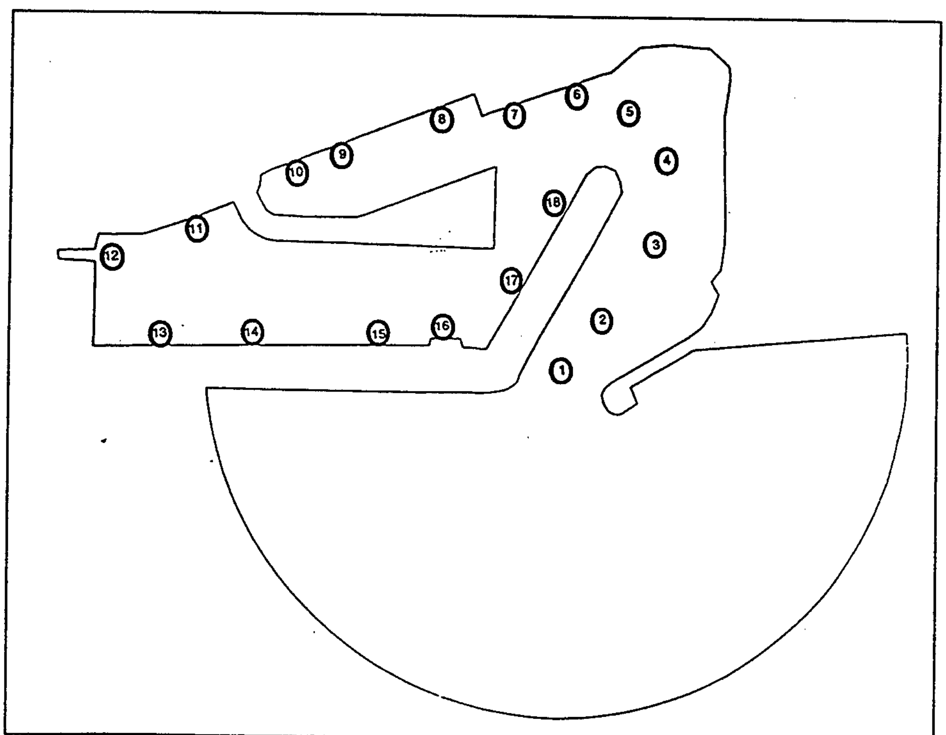

Figure B4. Output basin locations for Proposed Plans 6 and 6a

Appendix C HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria

Table C1
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Existing Plan

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	17	1.09	9.00	0.55	1.99	17
	15	1.00	7.91	0.55	1.82	7
	9	1.01	7.81	0.45	2.26	18
157.5	20	1.02	5.31	0.56		17
	17	1.01	5.00	0.55		7
	15	1.01	4.51	0.56		7
	13	1.01	3.91	0.56		7
	11	1.00	4.61	0.48		17
	9	1.02	4.21	0.48		18
180.0	20	1.02	4.71	0.56	1.83	17
	17	1.02	4.11	0.56	1.80	7
	15	1.00	3.71	0.59	1.70	7
	13	1.02	3.51	0.58	1.75	7
	11	1.02	3.81	0.50	2.01	18
	9	1.02	3.81	0.50	2.05	17
202.5	17	1.00	5.31	0.57	1.75	7
	15	1.01	4.91	0.61	1.67	7
	13	1.00	4.71	0.59	1.69	7
	11	1.01	4.91	0.53	1.92	7
	9	1.01	4.91	0.51	1.96	17
225.0	15	1.01	5.71	0.66	1.54	7
	13	1.01	5.61	0.62	1.62	7
	11	1.00	5.81	0.56	1.80	18
	9	1.00	5.51	0.55	1.81	17
247.5	15	1.01	8.00	0.60	1.68	7
	13	1.01	7.71	0.65	1.54	7
270.0	*					

(Continued)

Table C1 (Concluded)
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Existing Plan

2-ft Criterion						
135.0	*					
157.5	20 17 15 13 11 9	2.02 2.02 2.04 2.03 2.03 2.01	6.11 5.91 5.71 5.41 5.81 5.11	0.97 0.93 0.89 0.82 0.78 0.78	2.07 2.18 2.29 2.48 2.61 2.60	1 1 1 1 1 1
180.0	20 17 15 13 11 9	2.01 2.01 2.00 2.00 2.02 2.01	5.31 5.00 4.91 4.71 4.71 4.71	0.97 0.91 0.88 0.85 0.81 0.79	2.07 2.20 2.25 2.35 2.49 2.53	1 1 1 1 1 1
202.5	17 15 13 9	2.02 2.03 2.01 2.00	6.61 6.61 6.41 6.11	0.93 0.90 0.87 0.82	2.18 2.24 2.31 2.44	1 1 1 1
225.0	15 13 9	2.01 2.00 2.02	7.81 7.41 6.91	0.95 0.93 0.89	2.11 2.15 2.28	1 1 1
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C2
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 1

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C3
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 1a

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	*					
180.0	17	1.02	9.00	0.26	3.97	11
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C4
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 1b

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	17	1.07	9.00	0.32	3.32	11
	15	1.01	7.81	0.32	3.13	11
180.0	20	1.09	9.00	0.31	3.50	11
	17	1.01	7.31	0.31	3.21	11
	15	1.00	7.11	0.31	3.26	11
	13	1.01	6.91	0.29	3.45	11
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C5
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 2

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	9	1.01	7.91	0.44	2.29	7
157.5	20	1.01	7.81	0.38	2.66	7
	17	1.00	7.81	0.35	2.88	8
	15	1.01	7.00	0.36	2.81	23
	13	1.00	5.21	0.42	2.38	23
	11	1.00	5.21	0.43	2.33	7
	9	1.01	4.51	0.44	2.29	7
180.0	20	1.01	7.00	0.37	2.73	7
	17	1.00	6.31	0.36	2.77	8
	15	1.00	5.61	0.39	2.57	23
	13	1.01	4.81	0.42	2.40	23
	11	1.01	4.51	0.42	2.39	7
	9	1.02	4.31	0.44	2.32	7
202.5	17	1.09	9.00	0.37	2.98	8
	15	1.01	7.41	0.40	2.52	23
	13	1.01	6.61	0.42	2.38	23
	9	1.00	5.71	0.44	2.28	7
225.0	9	1.00	6.91	0.44	2.28	7
247.5	*					
270.0	*					

(Continued)

Table C5 (Concluded)
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 2

2-ft Criterion						
135.0	9	2.00	6.91	1.00	2.00	1
157.5	20	2.11	9.00	0.69	3.06	1
	17	2.03	7.11	0.77	2.63	1
	15	2.03	6.11	0.83	2.44	1
	13	2.01	4.71	0.93	2.16	1
	11	2.03	4.61	0.98	2.07	1
	9	2.04	4.00	1.00	2.04	1
180.0	20	2.02	7.21	0.72	2.81	1
	17	2.03	5.61	0.82	2.46	1
	15	2.01	5.00	0.87	2.30	1
	13	2.01	4.31	0.93	2.15	1
	11	2.03	3.91	0.98	2.07	1
	9	2.02	3.81	0.98	2.05	1
202.5	17	2.00	7.31	0.83	2.41	1
	15	2.02	6.71	0.89	2.28	1
	13	2.00	5.91	0.94	2.12	1
	9	2.02	5.11	0.99	2.04	1
225.0	15	2.02	8.00	0.93	2.16	1
	13	2.00	7.21	0.96	2.09	1
	9	2.02	6.21	0.99	2.05	1
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C6
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 3

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C7
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 6

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	20	2.02	6.61	0.90	2.24	1
	17	2.01	6.51	0.84	2.40	1
	15	2.02	6.41	0.79	2.55	1
	13	2.01	6.00	0.73	2.76	1
	9	2.00	6.81	0.58	3.47	1
180.0	20	2.00	5.71	0.90	2.22	1
	17	2.03	5.71	0.81	2.51	1
	15	2.01	5.51	0.79	2.53	1
	13	2.01	5.31	0.76	2.65	1
	11	2.00	5.51	0.68	2.91	1
	9	2.00	5.81	0.64	3.13	1
202.5	17	2.01	7.41	0.82	2.44	1
	15	2.02	7.31	0.81	2.48	1
225.0	15	2.12	9.00	0.87	2.43	1
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Table C8
HARBD-SHALWV Wave Heights Exceeding HQUSACE Criteria, Plan 6a

Deepwater Direction (deg az.)	Deepwater Period (sec)	Height (ft)	Deepwater Height (ft)	HARBD Amp. Factor	SHALWV Height (ft)	Basin Number
1-ft Criterion						
135.0	*					
157.5	*					
180.0	*					
202.5	*					
225.0	*					
247.5	*					
270.0	*					
2-ft Criterion						
135.0	*					
157.5	20	2.00	5.71	1.03	1.94	1
	17	2.00	5.71	0.95	2.11	1
	15	2.03	5.71	0.89	2.28	1
	13	2.01	5.51	0.79	2.53	1
	9	2.02	5.91	0.67	3.00	1
180.0	20	2.03	5.11	1.01	1.99	1
	17	2.02	5.00	0.90	2.20	1
	15	2.03	5.00	0.88	2.30	1
	13	2.02	4.91	0.80	2.45	1
	11	2.03	5.11	0.75	2.70	1
	9	2.03	5.21	0.72	2.81	1
202.5	17	2.03	6.61	0.92	2.18	1
	15	2.01	6.61	0.88	2.24	1
	13	2.01	6.61	0.85	2.38	1
	9	2.00	6.71	0.75	2.68	1
225.0	15	2.01	7.91	0.92	2.13	1
	13	2.02	7.71	0.88	2.23	1
247.5	*					
270.0	*					

*Deepwater wave heights between 1-9 ft do not exceed HQUSACE criteria for this condition.

Appendix D
Percent Occurrence of Wave Height Versus Direction

Table D1
Percent Occurrence of Wave Height Versus Direction Existing Plan - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00		0.16	3.25					3.41
4.01-5.00		3.15	8.97	1.01				13.14
5.01-6.00		2.09	5.32	1.24	0.17			8.80
6.01-7.00		1.24	2.04	0.52	0.31			4.11
7.01-8.00	0.06	1.14	0.41	0.03	0.17	0.01		1.84
8.01-9.00	0.14	0.31	0.28	0.07	0.03	0.03		0.87
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.20	8.13	20.28	2.91	0.72	0.08	0.45	32.77

Table D2
Percent Occurrence of Wave Height Versus Direction Existing Plan - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00			2.44					2.44
5.01-6.00		1.18	5.30					6.48
6.01-7.00		1.24	2.04	0.28	0.01			3.57
7.01-8.00		1.14	0.41	0.03	0.07			1.66
8.01-9.00		0.31	0.28	0.07	0.03			0.69
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	3.91	10.47	0.42	0.14	0.03	0.45	15.43

Table D3
Percent Occurrence of Wave Height Versus Direction Plan 1 - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D4
Percent Occurrence of Wave Height Versus Direction Plan 1 - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D5
Percent Occurrence of Wave Height Versus Direction Plan 1a - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00			0.17					0.17
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.17	0.03	0.03	0.03	0.45	0.76

Table D6
Percent Occurrence of Wave Height Versus Direction Plan 1a - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D7
Percent Occurrence of Wave Height Versus Direction Plan 1b - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00			0.03					0.03
7.01-8.00		0.03	0.37					0.40
8.01-9.00		0.28	0.28					0.55
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.34	0.69	0.03	0.03	0.03	0.45	1.58

Table D8
Percent Occurrence of Wave Height Versus Direction Plan 1b - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D9
Percent Occurrence of Wave Height Versus Direction Plan 2 - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00		0.87	1.61					2.48
5.01-6.00		0.93	3.07	0.03				4.03
6.01-7.00		0.25	1.78	0.05	0.01			2.09
7.01-8.00	0.04	0.41	0.41	0.02				0.90
8.01-9.00	0.07	0.31	0.28	0.07	0.03			0.76
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.11	2.81	7.15	0.21	0.08	0.03	0.45	10.84

Table D10
Percent Occurrence of Wave Height Versus Direction Plan 2 - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00			0.29					0.46
4.01-5.00		0.17	3.66					5.76
5.01-6.00		2.11	4.82	0.10				5.89
6.01-7.00	0.11	0.97	1.86	0.19	0.03			2.82
7.01-8.00	0.21	0.62	0.41	0.03	0.05			1.81
8.01-9.00	0.07	1.10	0.28	0.07	0.03			0.76
9.01+		0.31		0.03	0.03	0.03	0.45	0.59
TOTAL	0.39	5.31	11.32	0.44	0.15	0.03	0.45	18.09

Table D11
Percent Occurrence of Wave Height Versus Direction Plan 3 - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D12
Percent Occurrence of Wave Height Versus Direction Plan 3 - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D13
Percent Occurrence of Wave Height Versus Direction Plan 6 - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D14
Percent Occurrence of Wave Height Versus Direction Plan 6 - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00		0.03	3.10					3.13
6.01-7.00		0.74	2.04					2.78
7.01-8.00		1.14	0.41	0.03				1.58
8.01-9.00		0.31	0.28	0.07	0.03			0.69
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	2.26	5.83	0.13	0.07	0.03	0.45	8.78

Table D15
Percent Occurrence of Wave Height Versus Direction Plan 6a - Wave Heights Exceeding 1 ft in Berthing Areas

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00								
5.01-6.00								
6.01-7.00								
7.01-8.00								
8.01-9.00								
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	0.03	0.0	0.03	0.03	0.03	0.45	0.59

Table D16
Percent Occurrence of Wave Height Versus Direction Plan 6a - Wave Heights Exceeding 2 ft in Channel

Deepwater Wave Height, ft	Deepwater Wave Direction (deg azimuth)							
	135.0	157.5	180.0	202.5	225.0	247.5	270.0	Total
3.01-4.00								
4.01-5.00			0.96					0.96
5.01-6.00		0.76	5.28					6.04
6.01-7.00		1.24	2.04	0.26				3.54
7.01-8.00		1.14	0.41	0.03	0.04			1.63
8.01-9.00		0.31	0.28	0.07	0.03			0.69
9.01+		0.03		0.03	0.03	0.03	0.45	0.59
TOTAL	0.0	3.49	8.97	0.40	0.11	0.03	0.45	13.45

Appendix E
HARBD Wave Amplification Factors, Harbor Oscillations

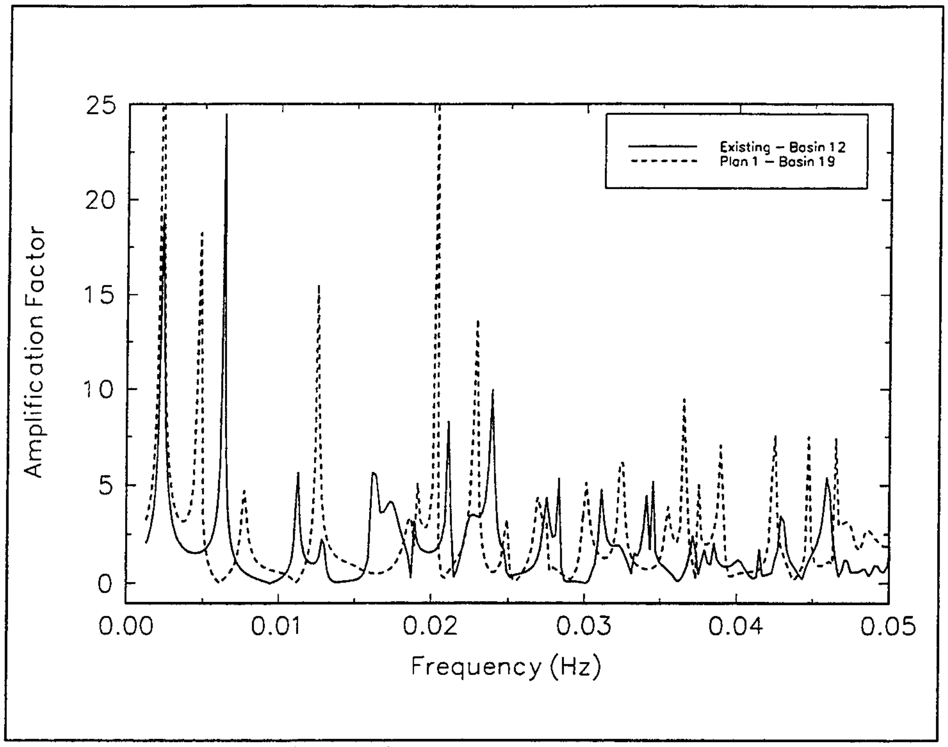

Figure E1. Wave amplification factor, west end, Plan 1 vs Existing Plan

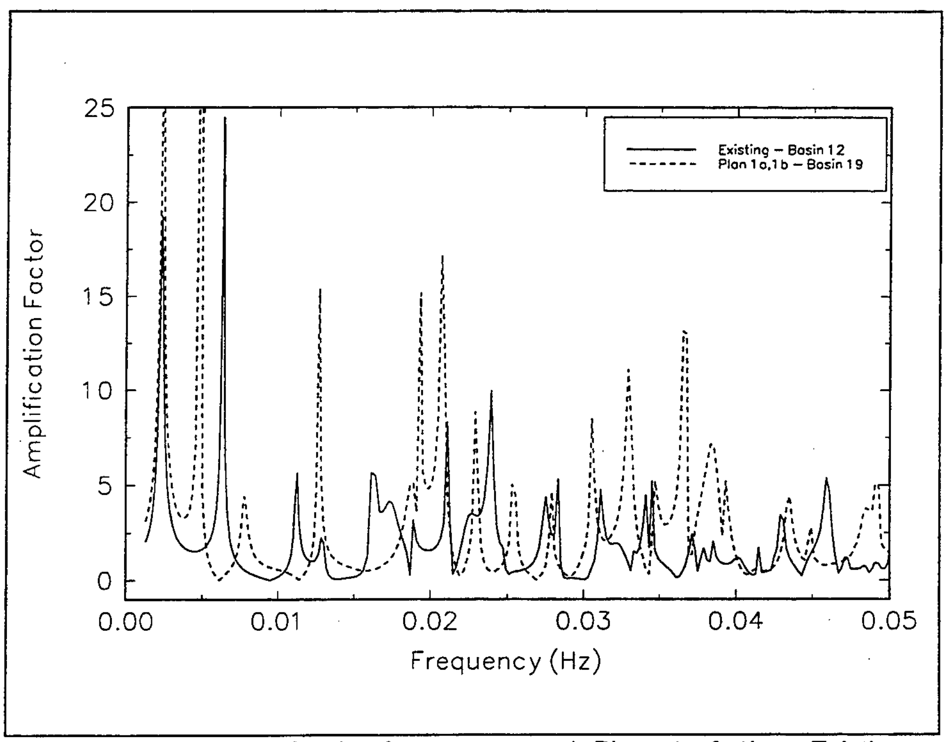

Figure E2. Wave amplification factor, west end, Plans 1a & 1b vs Existing Plan

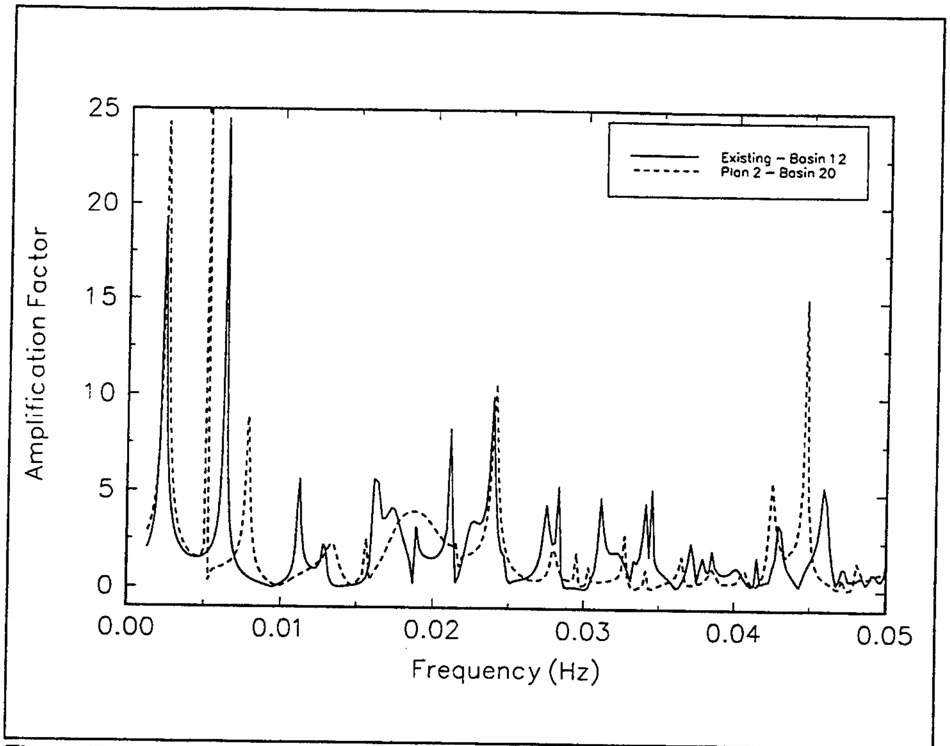

Figure E3. Wave amplification factor, west end, Plan 2 vs Existing Plan

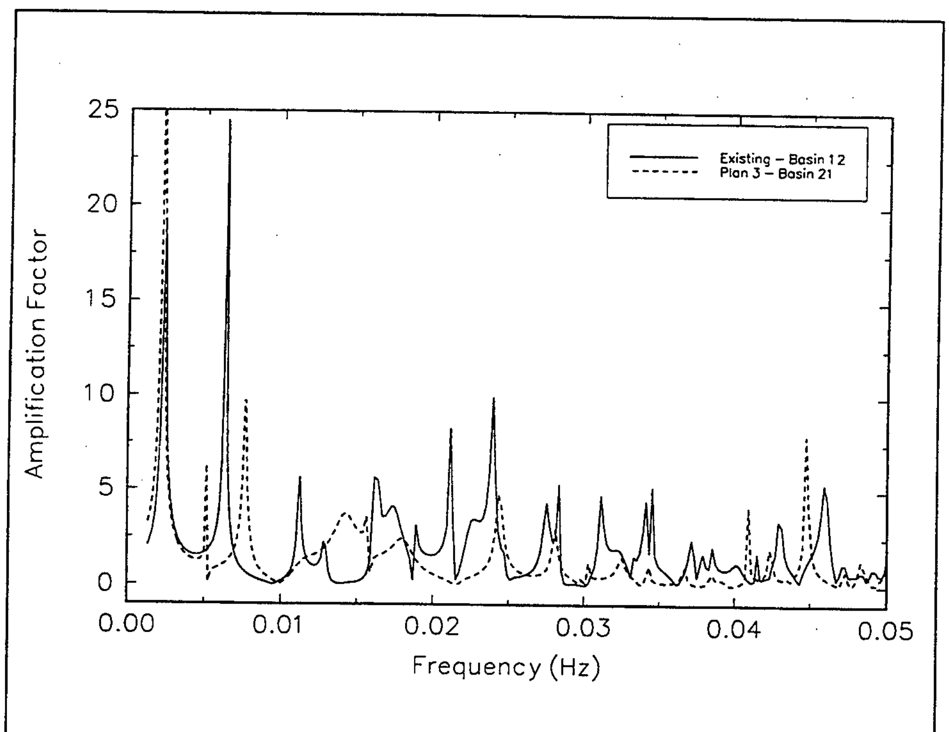

Figure E4. Wave amplification factor, west end, Plan 3 vs Existing Plan

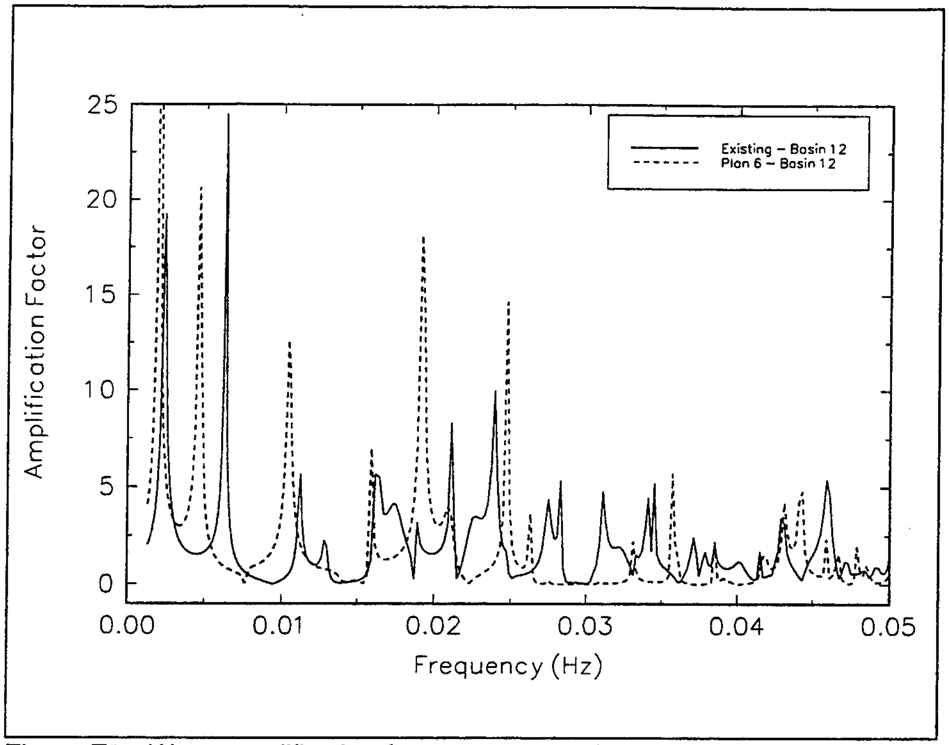

Figure E5. Wave amplification factor, west end, Plan 6 vs Existing Plan

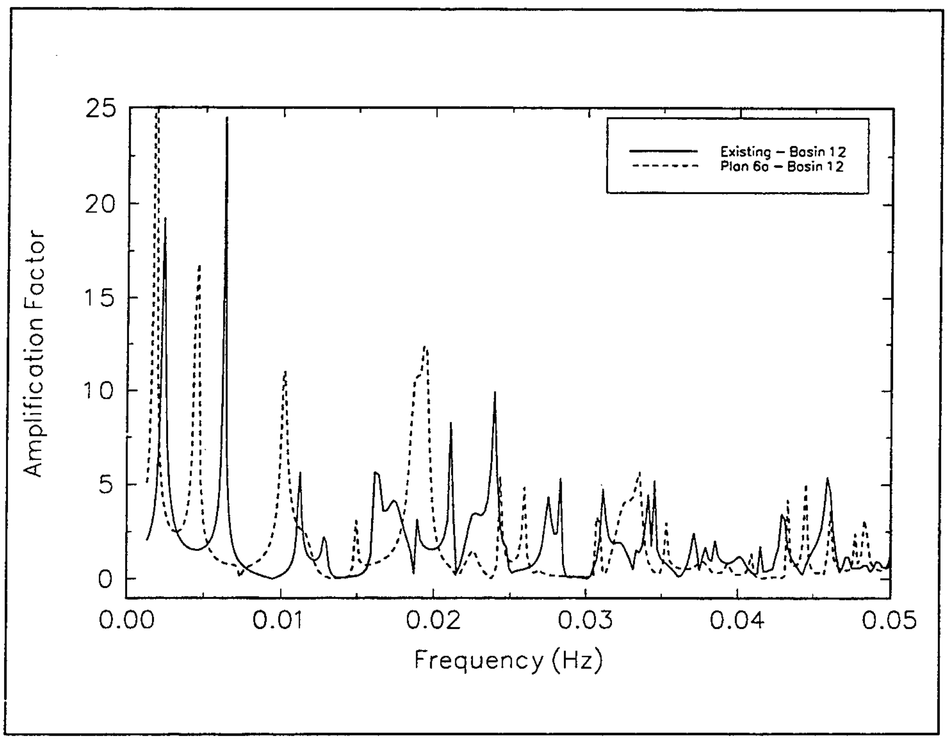

Figure E6. Wave amplification factor, west end, Plan 6a vs Existing Plan

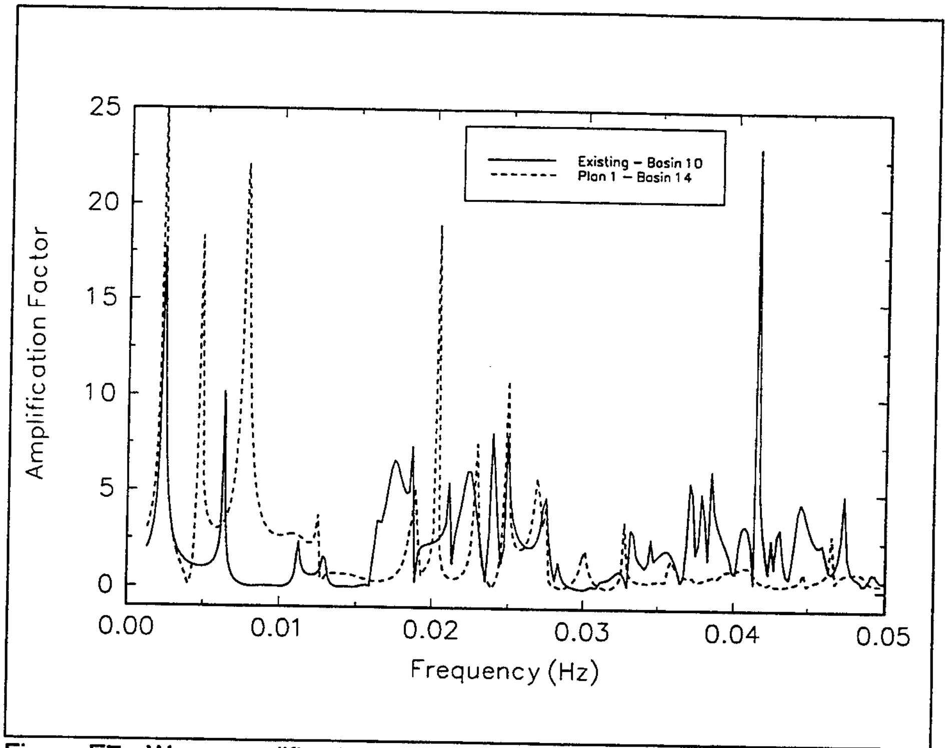

Figure E7. Wave amplification factor, north boundary, Plan 1 vs Existing Plan

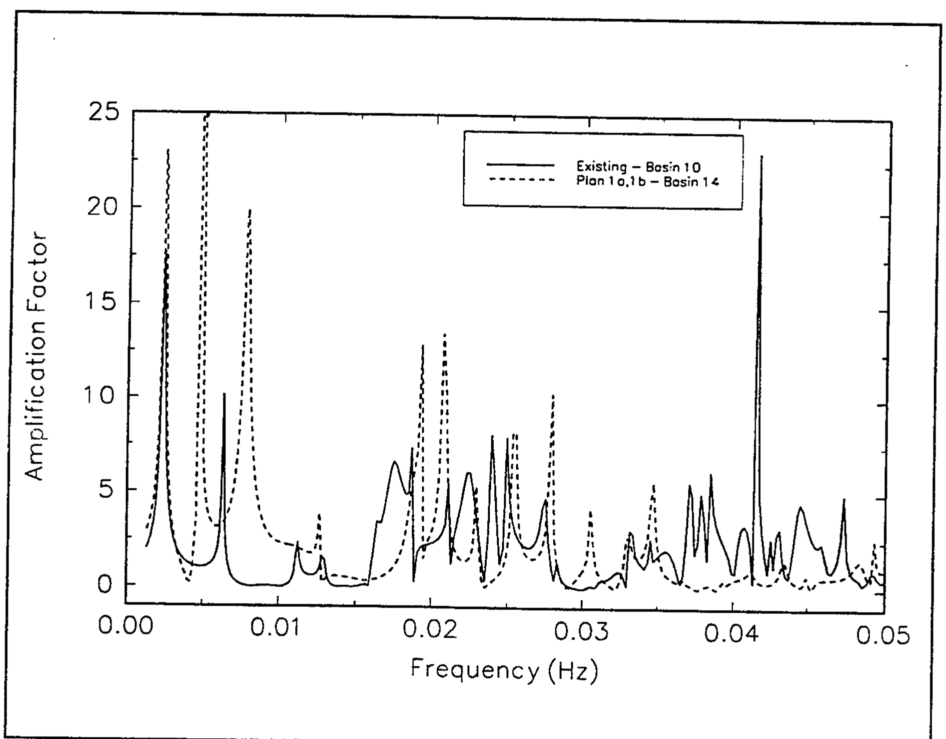

Figure E8. Wave amplification factor, north boundary, Plans 1a & 1b vs Existing Plan

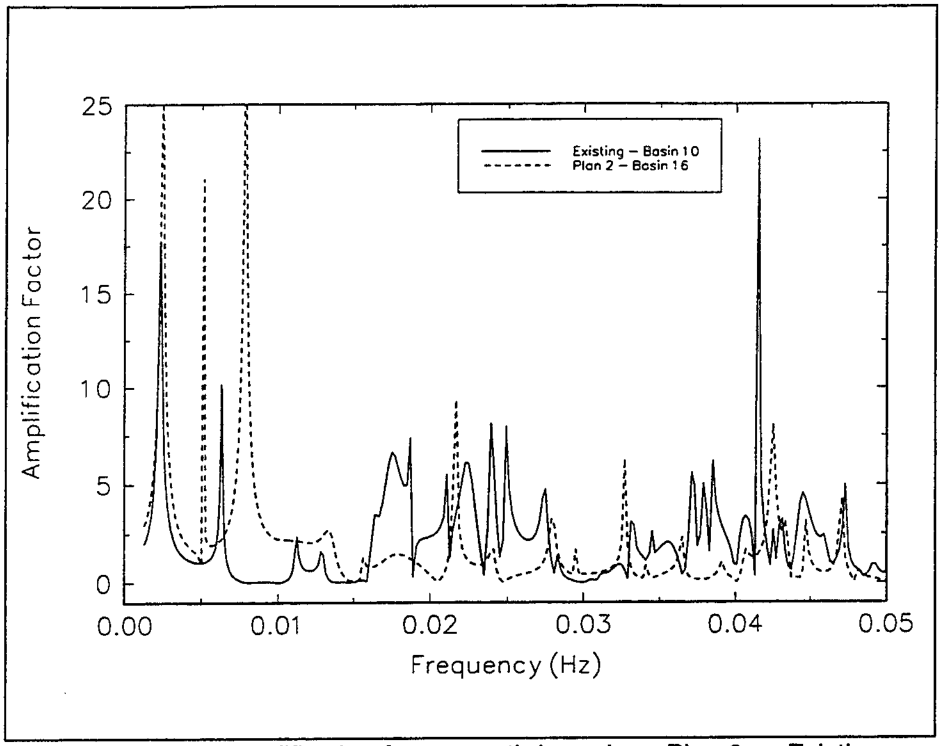

Figure E9. Wave amplification factor, north boundary, Plan 2 vs Existing Plan

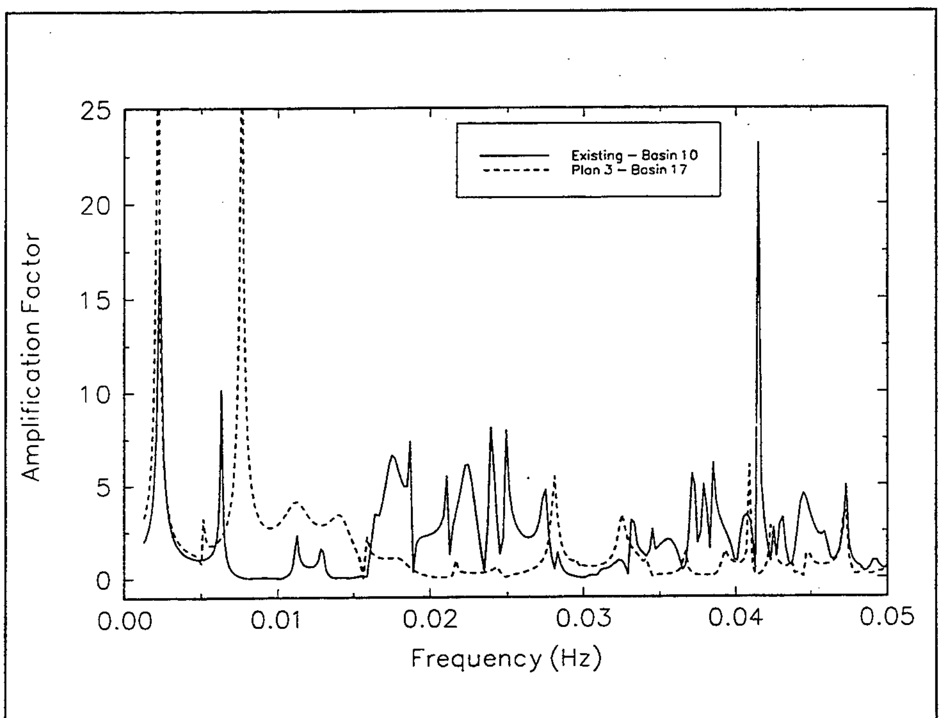

Figure E10. Wave amplification factor, north boundary, Plan 3 vs Existing Plan

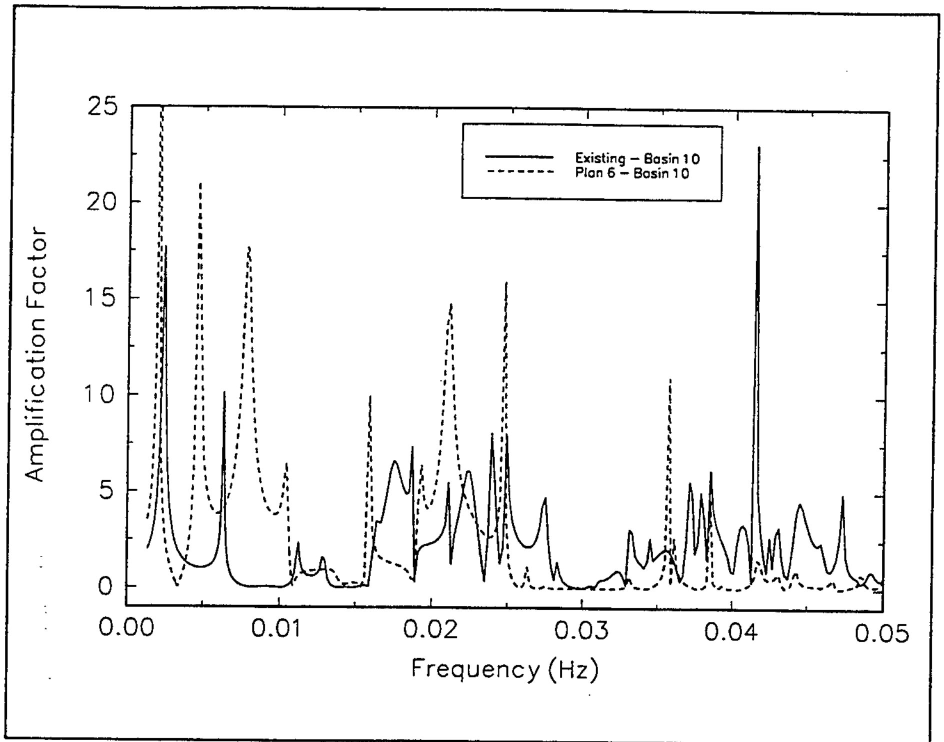

Figure E11. Wave Amplification factor, north boundary, Plan 6 vs Existing Plan

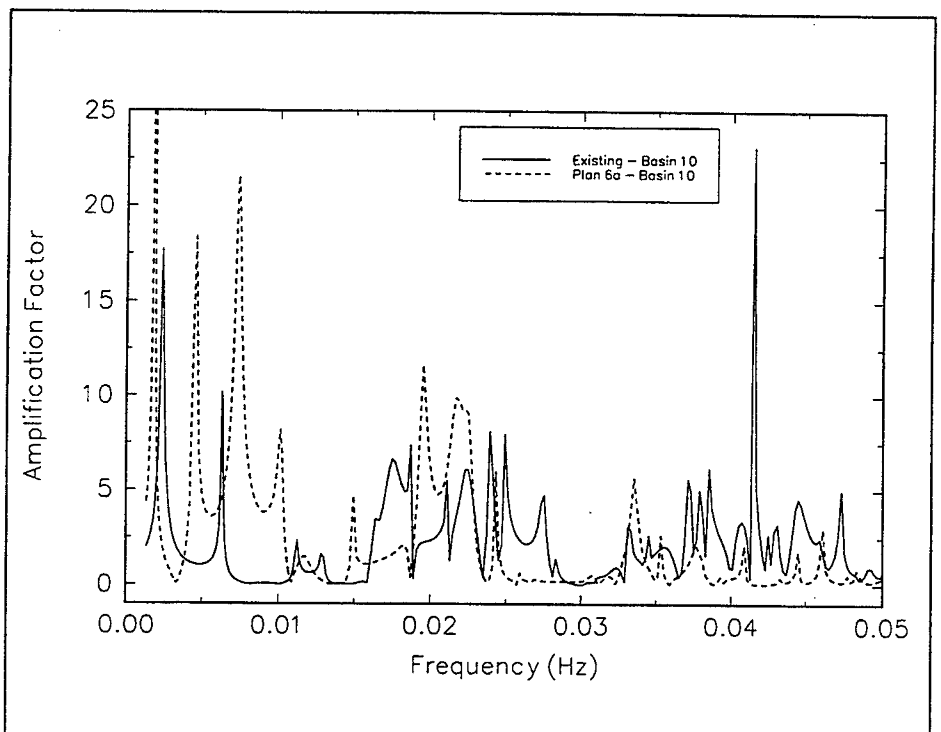

Figure E12. Wave amplification factor, north boundary, Plan 6a vs Existing Plan

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13. ABSTRACT *(Maximum 200 words)*

The U.S. Army Engineer Division, Pacific Ocean (CEPOD), requested that the U.S. Army Engineer Waterways Experiment Station numerically study the wave response of proposed improvement plans to the small boat harbor at Maalaea, Maui, Hawaii. This study is an update of earlier studies to assess the wave response of the existing harbor and seven proposed modification plans. Deepwater wave data used in earlier studies were derived from measurements obtained in the Monitoring of Completed Coastal Projects Program. The deepwater wave climate used for the current study was taken from a National Data Buoy Center buoy located southwest of the island of Lanai. The availability of deepwater data nearer the vicinity of Maalaea Harbor, combined with improved harbor modeling technology, significantly improves the validity of overall results.

The results of this study show that all of the proposed harbor plans showed some degree of improvement over the existing plan in providing protection from incident wind waves and swell to berthing areas. However, Plans 2, 6, and 6a are potentially more hazardous to navigation during high wave conditions due to a constricted harbor entrance exposed to wind waves and swell.

14. SUBJECT TERMS

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Harbor oscillations	Wave refraction
Hawaii	Wave response

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